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THE UNIVERSITY OF ALBERTA

A STUDY ON RIPRAP DESIGN
FOR BRIDGE PIERS

BY

MAHBOOB ELAHI QUAZI

A THESIS

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
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The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled, A STUDY ON RIPRAP DESIGN FOR BRIDGE PIERS, submitted by Mahboob Elahi Quazi in partial fulfilment of the requirements for the degree of Master of Science.

ABSTRACT

The critical stability of coarse granular materials used as riprap around a bridge pier was experimentally studied. A comparative analysis of the results of this study with the data on initiation of motion on plane beds indicates that the presence of a pier lowers the mean channel velocities required to move the material. A graphic correlation was developed to show the extent of this change in the mean velocity, using the data of this study and the extensive sediment transport data for zero or low charge. In the above correlation a form of the densimetric Froude number was related to the ratio between the depth of flow and the characteristic size of the riprap material. A design curve was derived, which for a set of flow conditions provides two grain sizes; one that is stable on a plane bed and the other near a pier.

Riprap tests for this study were conducted around a pier of fixed shape and size, using several sizes of various types of materials as riprap for a range of flow conditions. A few tests were conducted to study qualitatively the effect of pier size and the angle of attack on the riprap stability.

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SYMBOLS

A	Experimental constant
a_o	Radius of the cylinder
a'	Lateral extent of the riprap around the pier (ft)
a	Regression coefficient
B	Width of the channel (ft)
b_1	Pier width (cm or ft)
b	Regression coefficient
c	Concentration of bed-material discharge by weight of total discharge (ppht)
d_{se}	Equilibrium scour depth (ft)
d_s	Maximum scour depth (ft)
d_p	Pier diameter (ft)
d	Depth of flow (ft)
D	Representative size of the approach bed
D_m	The mean particle size (ft or mm)
D_{15}, D_{50}, D_{85}	Grain sizes of which the given percent by weight of the material is finer
D_w	Width of the riprap protection around the pier
D_p	Thickness of the protected layer (ft)
DG	Characteristic size of the riprap (ft or mm)
f	Lacey's silt factor (mm)
F	Froude number ($VMC/\sqrt{gH}$)
Fo'	Critical Froude number
FR1	Mobility number ($\rho_f VMC^2/\gamma'_S DG$)

FR'	$g/\sqrt{(SG-1)g DG^3}$
FRD	$VMC/\sqrt{g(SG-1)DG}$
FRD'	$VMC/\sqrt{g(SG-1)H}$
g	Acceleration due to gravity (ft/sec ²)
G	Coefficient for pier shape
H	Depth of flow (cm or ft)
h _P	Pier height (ft)
k	Experimental constant
L _P	Length of the pier (ft)
M	Experimental coefficient
N, N', N''	Sediment numbers (Cartsen)
p	Distance of the top of the riprap from streambed (ft)
q ₁	Discharge (liters/sec)
q _c	Critical discharge (liters/sec)
Q	Discharge (ft ³ /sec)
Q _m	Maximum flood discharge (ft ³ /sec) (Inglis)
q	Discharge per unit width (Q/B)
RN	Pier Reynold number (VMC W/v)
S _T	Terminal scour depth (cm)
SL	Limiting scour depth (cm)
S _P	Shape factor for pier nose
SG	Specific gravity
S	Slope of the energy-grade line
t _s	Maximum scour depth (cm)
T	Riprap thickness (ft)

t	Thickness of the riprap layer (ft)
t_o	Time of observation (sec) (Yalin)
V	Mean velocity (ft/sec)
V_{PC}	Velocity near the pier at riprap failure (ft/sec or cm/sec)
V_{MC}	Mean velocity of the flow at the instant of first observed instability of the riprap (cm/sec)
V_*	Shear velocity (ft/sec)
W	Width of the pier (ft)
W^*	Weight of the stone (lbs)
X_b, X_r	Parameters for the shape of the approach bed and the riprap materials
X	Independent variable
Y	Dependent variable
γ_s, γ	Specific weight of the sediment and the fluid (lb/ft ³)
$\Delta\Gamma$	Change in circulation
θ	Angle of attack (degrees)
E_p	Measure of pier's surface roughness
δ_b, δ_r	Gradation parameter for the approach bed and the riprap material
ρ_r, ρ_b, ρ_f	Mass density of the riprap material, bed material and of the flow (slugs/ft ³)
ν	Kinematic viscosity of the fluid (ft ² /sec)
γ'_s	Buoyant unit weight $\gamma'_s = g(\rho_r - \rho_f)$ (lbs/ft ³)
TU	Mean bed shear stress (lb/ft ²)

CHAPTER I

INTRODUCTION

1-1 General

Hydraulic engineers have long been concerned with localized scour around bridge piers in natural streams. Over the past two decades model studies have provided sufficient empirical knowledge to replace rules of thumb used previously in designing bridge pier foundations. Scour in general is defined in Civil Engineering Glossary (Blaisdell et al. 1962) as "the erosive action of running water in excavating and carrying away material from bed and banks." Deep scour holes formed by flowing water around bridge piers have resulted in many bridge failures (Neill, 1964). The logical choice for an engineer, with the present knowledge of bridge pier scour, is to design and place the bridge pier foundation below the deepest scour expected. This design may result in a very deep foundation; an alternative is to devise a way to reduce the depth of foundation.

1-2 Definition of Riprap

Out of the many methods of reducing scour around bridge piers, one is to armour the bed around the pier. This armouring is achieved by placing loose riprap on the stream bed around the pier. The technical definition of riprap (Blaisdell et al. 1962) is 'broken stone placed on earth surfaces for their protection against the action of water.' However, in this study natural gravels and other materials are

also referred to as riprap.

1-3 The Problem

The problem posed for experimental study was:

Given certain flow conditions with a bridge pier placed in the channel, determine the size of the riprap that would prevent any local scour around the pier.

1-4 Scope of the Study

Considering the complex nature of the local scour problem and the limited scope of this study, the experimental investigation were limited to:

- (i) Clear water scour case, where there is no sediment movement in the channel approaching the pier.
- (ii) Piers with a standard round nose (Fig. 1-1)
- (iii) Riprap tests for all the materials around a 2.5" x 6" pier aligned parallel to the flow.
- (iv) A few tests around piers of 1" x 6" and 3" x 6" size and round nosed shape.
- (v) Riprap materials of six sizes of natural gravels, one size each of glass balls and cellulose acetate balls.
All material tested were of uniform size.
- (vi) A few tests with a 2.5" x 6" pier

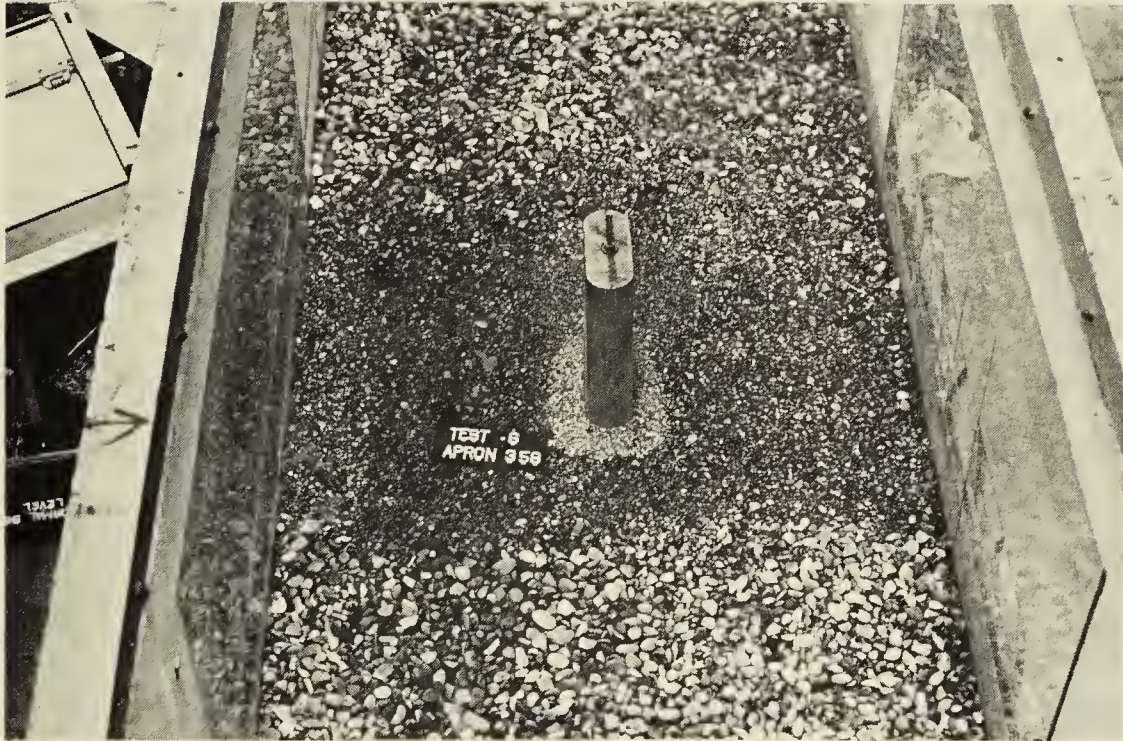


FIGURE 1-1 PHOTOGRAPH OF THE TEST PIER

aligned at 15° and 25° to the flow.

- (vii) The flow regime was limited to $FR < 1$.
- (viii) The maximum riprap size tested was limited by the pier width to stone size ratio (W/D_G) and the Froude number (FR). Tests were limited to $W/D_G > 4.0$ and $FR < 0.60$. For $W/D_G < 4$ and $FR > 0.60$ failure of the riprap occurs at the back of the pier.
- (ix) Removal of the underlying materials around the pier by suction and the consequent failure of the riprap was prevented by providing the same material as the riprap up to the flume bed around the pier.
- (x) After a few tests a program to measure the bed shear stress distribution around the pier using a 10 mm propeller-meter was abandoned due to discouraging results.

CHAPTER II

SURVEY OF LITERATURE

2-1 General

This chapter presents a summary of the previous work done on various aspects of bridge pier scour problem. An effort has been made to make the presentation in a logical way, discussing the mechanics of flow as well as the results of empirical studies on pier scour. Such an approach is necessary for resolving the pier protection problem.

Neill (1964) summarizes results of most of the model studies done on pier scour. Shen (1971) presents a comprehensive review of all aspects of scour near piers. Shen, Schneider and Karaki (1966) present a thorough review of methods of reducing pier scour and the results of their own studies on the problem. Posey et al. (1951) and Sousa-Pinto (1959) deal with riprap protection in detail.

The literature review is presented under the following main headings:

1. Flow structure around a pier.
2. Model studies on bridge pier scour.
3. Methods of reducing bridge pier scour.
4. Riprap protection.

2-2 Mechanics of Local Scour

Characteristics of flow around a bridge pier have been recorded by Tison (1940), Neill (1964), Laursen and Toch (1953) and Posey et al. (1951). All these studies point out the existence of a strong diving

flow at the nose of the pier, which in their opinion, is the main reason for the localized scour.

Shen, Schneider and Karaki (1969) describe the dominant features of flow around a pier, as the large scale eddy structure which develops around the pier. Depending upon the type of pier and free stream conditions this eddy structure could be composed of one or more of the three basic systems: the horse-shoe vortex system, the wake-vortex system and the trailing-vortex system.

The horse-shoe vortex system is formed due to the separation of the three-dimensional boundary layer ahead of the pier. The mechanism triggering this separation of the boundary layer is the pressure field induced by the pier. Schlichting (1961) gives a general introduction to the three-dimensional boundary layer ahead of a cylinder which is very similar to the case of a pier. The stagnation plane is the plane of symmetry of Fig. 2-1. The velocity profiles on the plane of symmetry are collateral (i.e. all velocity vectors lie on a flat plane). The separation of the plane of symmetry boundary layer occurs at a point marked S as the layer cannot sustain forward motion under the pressure rise due to the stagnation. Away from the plane of symmetry this pressure rise causes the boundary layer to become skewed as shown in Fig. 2-1. The separation of this skewed layer also occurs, but in a different manner (Taylor, 1959). Johnston (1960), Hornung and Joubert (1963) investigated the characteristics of the mean velocity profiles of the three-dimensional skewed boundary layers. Toomre (1959) gives a theoretical solution for the viscous secondary flow ahead of an infinite cylinder in a uniform parallel shear flow. Moore and Masch (1963) applied Toomre's

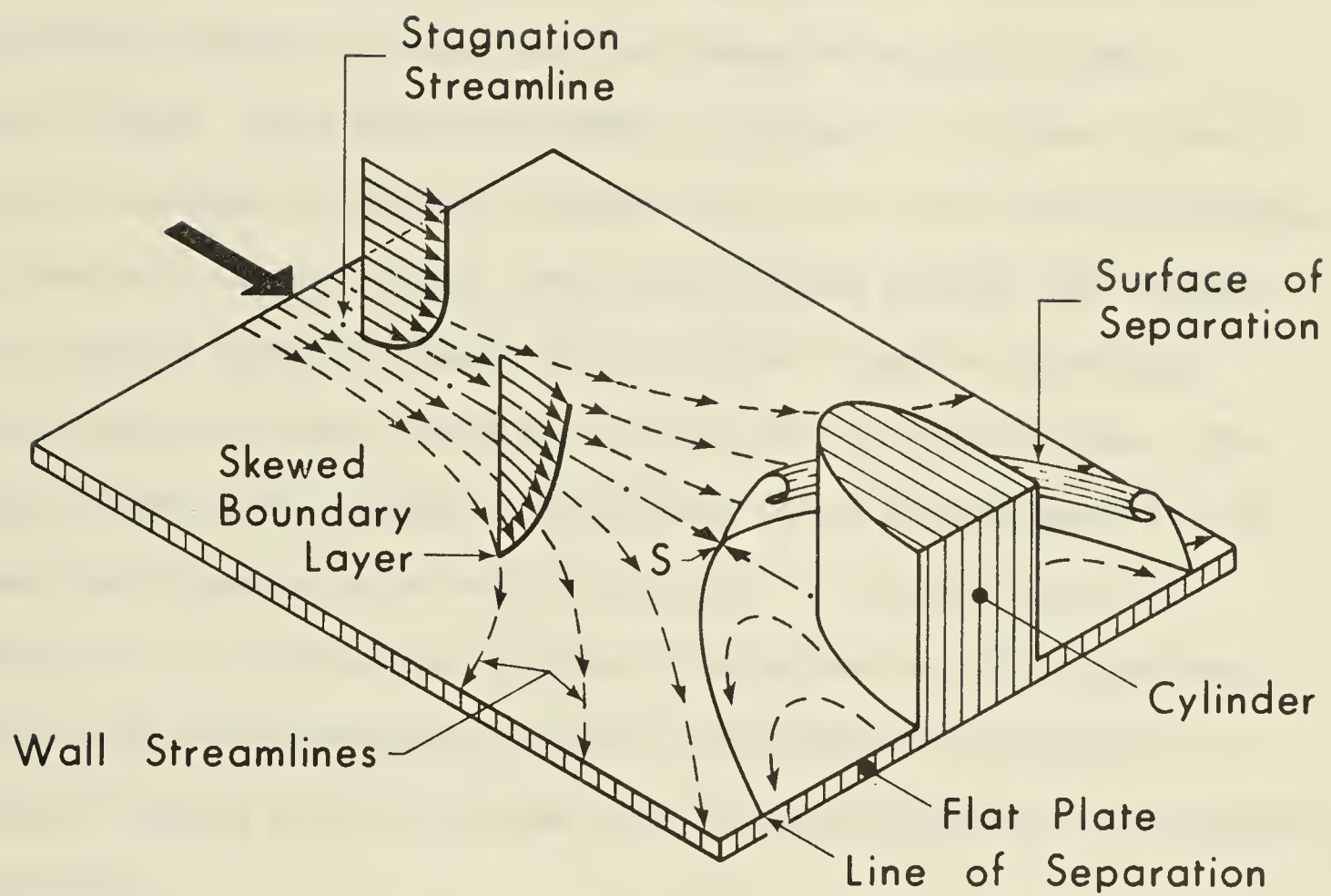


FIGURE 2-1 CHARACTERISTICS OF FLOW AHEAD OF A PIER

(1959) solution to the case of a bridge pier.

The wake-vortex system is formed by the separation of shear layers generated on the pier surface. Characteristics of such a wake for a circular cylinder have long interested the experts in fluid mechanics (Morkovin, 1964). In brief, the location of change to turbulence and the manner of onset of turbulence, in the wake behind a circular cylinder, changes with the change in Reynold's number (Bloor, 1963). This Reynold's number is defined on cylinder diameter. With the increase in Reynold's number beyond 300, the location of change to turbulence in the cylinder wake starts moving towards the cylinder. At a critical Reynold's number of 3×10^5 the change to turbulence occurs before the shear layer has time to roll up into vortices. The manner in which this change to turbulence occurs also changes with the transition occurring in the separated layer. In early stages this transition to turbulence is supposed to occur due to distortion from large-scale three-dimensional effects, while after the critical Reynold's number this is believed to be caused by Tollmien - Schlichting instability.

The strength of vortices in the wake system depends on the shape and size of the pier. A stream-lined pier will create a relatively weak wake as compared to a blunt nosed pier. The Wake-vortex system creates additional problems in the design of the riprap for a bridge pier. At Reynold's numbers beyond the critical, a high degree of turbulence in the wake near the pier lifts up the riprap materials from the pier back, while the riprap on the rest of area around the pier is still stable.

The trailing-vortex system occurs only on completely submerged piers. It consists of one or more discrete vortices attached to the top

of the pier and extending downstream. Their existence is attributed to the presence of finite pressure difference between two surfaces meeting at a corner (Shen, Schneider and Karaki, 1969).

2-3 Approaches to the Study of Pier Scour

Local scour has been studied using dimensional analysis, concepts borrowed from hydraulics of mobile-bed channels and approximations from fluid mechanics.

One very important distinction in local scour is between the scour with no sediment supply and the scour with continuous sediment supply to the scour hole. Equilibrium scour depth is reached in the first case when no more material is moved out of the scour hole, and in the second case when the supply to the scour hole almost equals the amount of material being moved out of it (Laursen, 1970).

Perhaps the earliest known relations for predicting local scour at bridge piers are from the exponents of the regime theory. Inglis (1949) gives the maximum depth of scour at a pier as twice the Lacey's regime depth that is:

$$d_s = 2(0.47(Q_m/f)^{1/3}) = 2D_{\text{Lacey}} \quad (2-1)$$

where:

Q_m = Max. flood discharge (cfs)

d_s = Max. scour depth below highest flood level (ft)

f = Lacey's silt factor

$$= 1.76 \sqrt{D_m}$$

D_m = mean diameter of the bed sand (mm)

Blench (1969) recommends the same rule for the scour depth at a pier, but modifies the Lacey equation, introducing a bed factor and a side factor.

Laursen (1969) treated pier scour as a limiting case of scour in a long contraction with continuous sediment supply. In his own words (1960):

"A bridge crossing is in effect a long contraction fore-shortened to such an extreme that it has only a beginning and an end. The flow at the crossing cannot be considered uniform, but the solutions for the long contraction can be modified to describe the scour at bridge piers and abutments with the use of experimentally determined co-efficients."

Laursen (1963) applied a similar analysis to the case of the clear water scour at a pier.

Knezevic (1960) defined a critical discharge to start local scour at a pier. His proposed formula for maximum depth of scour at a pier, based on his experimental data is:

$$t_s = A \frac{(q_1 - q_c)}{H^{5/4} g^{3/4}} \quad (2-2)$$

where:

t_s = Max. scour depth (cm)

q_1 = Discharge (Liters per second)

q_c = Critical discharge (Liters per second)

H = Depth of flow (cm)

A = Experimental constant

Cartsen (1966) defined a sediment number for the pier scour as:

$$N = \frac{V}{\sqrt{(\rho_b / \rho_f - 1) g D}} \quad (2-3)$$

where:

N = Sediment number

V = Mean flow velocity (fps)

ρ_b, ρ_f = Mass density of the sediment and the fluid

D = Representative sediment size

Cartsen (1966) recommends:

$$N' = N''/2 \text{ (For a cylindrical pier)} \quad (2-4)$$

where:

N' = Sediment number for initiation of local scour

N'' = Sediment number when continuous sediment motion is present

Cartsen (1966) found that for local scour around a cylindrical pier the relation is

$$ST / 2a_o = 0.546 \left(\frac{N_s^2 - 1.64}{N_s^2 - 5.02} \right)^{5/6} \quad (2-5)$$

where:

$2a_o$ = Cylinder diameter

ST = Terminal scour depth

N_s = Sediment number

Tarapore (1967) analysed local scour by determining the velocity diffusion pattern in the hole and thereby its transporting capacity. General reporters in IAHR (1967) pointed out the inadequacies of this approach.

Roper, Schneider and Shen (1967) attempted to estimate the

strength of the horse-shoe vortex system formed at the upstream side of a blunt-nosed pier, for sub-critical flow conditions. They considered a flow with unspecified but continuous velocity distribution in the stagnation plane ahead of a blunt-nosed cylinder. By taking a control volume in this plane, they computed the change in circulation caused by the presence of the cylinder as:

$$\Delta\Gamma = a_o V \quad (2-6)$$

where

a_o = Radius of the cylinder

V = Average upstream flow velocity

Roper, Schneider and Shen (1967) argued that in the formation of horse-shoe vortex viscosity is an important parameter. In non-dimensional terms

$$\Delta\Gamma = f \frac{(2a_o V)}{v} = f (RN)$$

where:

RN = Pier Reynold's number

v = Kinematic viscosity

Finally, as the scour at the nose of the cylinder is caused by the horse-shoe vortex, the equilibrium scour depth can be related to RN as:

$$dse = g (RN) \quad (2-7)$$

where:

dse = Equilibrium depth of scour (ft)

By plotting all the available data they found a equation for maximum scour depth as:

$$dse = 0.00073 RN^{0.619} \quad (2-8)$$

Gradowcyk, Maggiolo and Folguera (1968) attempted to produce a mathematical model for localized scour around piers, based on the shallow water theory. Their solution holds true when:

- (i) The boundary layer does not separate ahead of the pier.
- (ii) The surface of separation around the pier is known.
- (iii) The stream-line $\psi = \text{const.}$ is known.

Govind Rao et al. (1970) tried to use the concept of the critical Froude number in local scour analysis. They formulated an equation for pier scour as:

$$\frac{SL}{H} = M \times F - F_o' \quad (2-9)$$

where:

SL = Limiting scour depth (cm)

H = Depth of flow (cm)

M = Experimentally determined co-efficient

F_o' = critical Froude number

They found, from their limited experimental data, relations for M and F_o' as:

$$M = G b_1^{1/2} + 1 \quad (2-10)$$

where:

b_1 = Pier width (cm)

G = Co-efficient depending on the shape of the pier

$$F_o' = 0.5 D_m^{1/3} \quad (2-11)$$

D_m = diameter of sand (mm)

2-4 Model Tests on Pier Scour

Most of the present insight into pier scour has been acquired through model tests. Usually the approach has been to place the model of a pier in a wide erodible bed channel and carefully study the effects of each variable on the local scour around the pier.

Model tests have been conducted by Tison (1940), Inglis (1949), Laursen and Toch (1953), Chabert and Engeldinger (1956), Varzeliots (1960) and Qureshi (1965).

The variables that effect the pier scour problem arise from the pier geometry, the geometry of its location, the flow and the bed-material characteristics. The following discussion presents a brief summary of the model test results.

- (i) Pier size and length:- The maximum scour depth at the pier nose is basically related to the pier width; the wider the pier, the deeper is the observed maximum scour depth at its nose. If laboratory tests are run using different pier sizes keeping everything else the same Fig. 2-2(a) presents the variation of maximum scour depth with the pier size. Pier length does not seem to have any effect on maximum scour depth for piers aligned parallel to flow (Varzeliots, 1960).
- (ii) Pier Shape:- For parallel aligned piers, nose shape does seem to have some effect on the maximum scour depth. From all the shapes lenticular shape gives the minimum scour depth under comparable conditions (Neill, 1964).
- (iii) Angle of Attack:- Scour is effected by the angle of attack

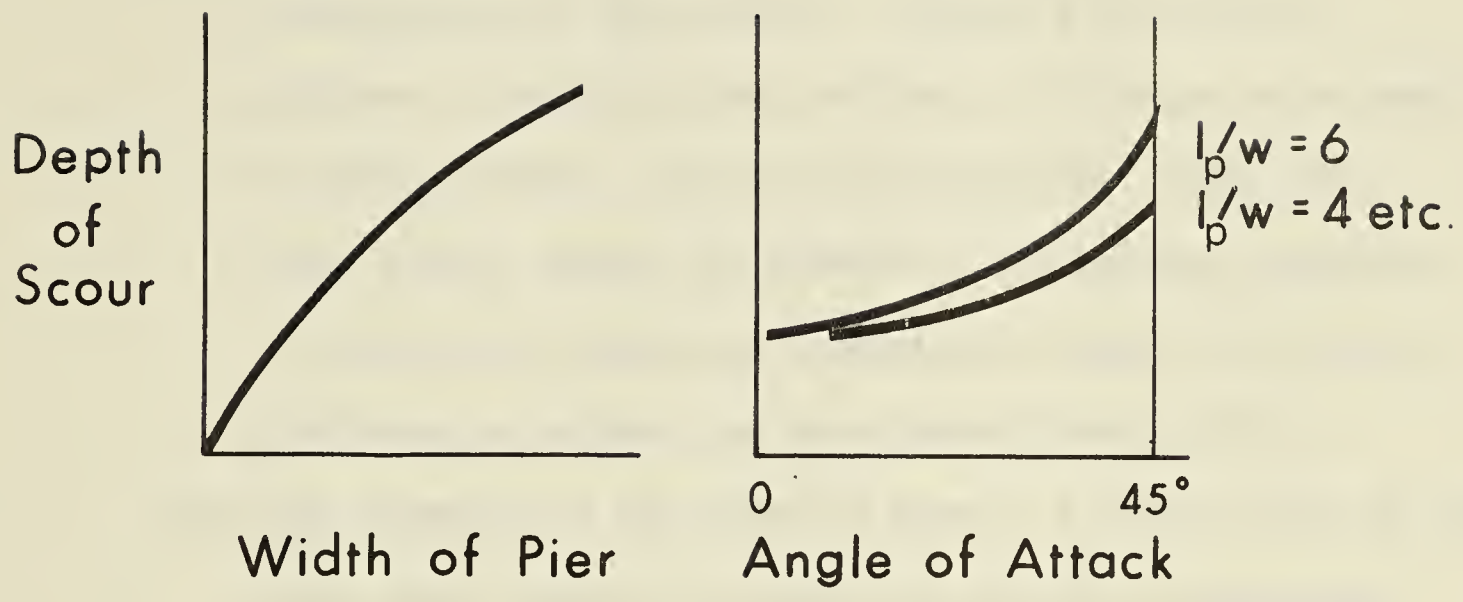


FIGURE 2-2(a) EFFECT OF PIER WIDTH AND ALIGNMENT ON DEPTH OF SCOUR

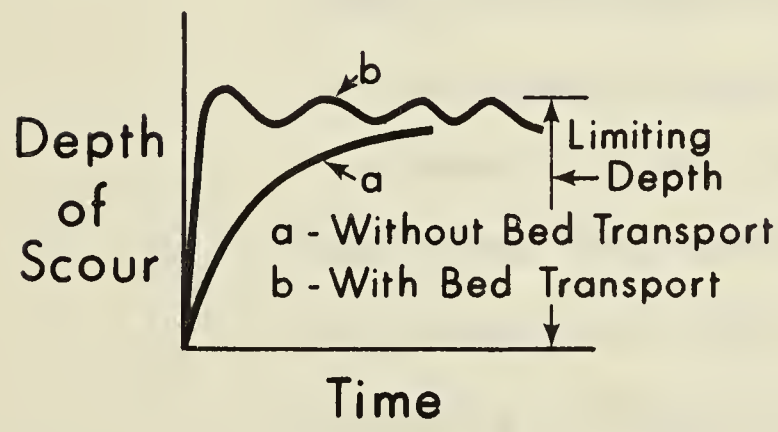


FIGURE 2-2(b) INCREASE OF SCOUR DEPTH WITH TIME

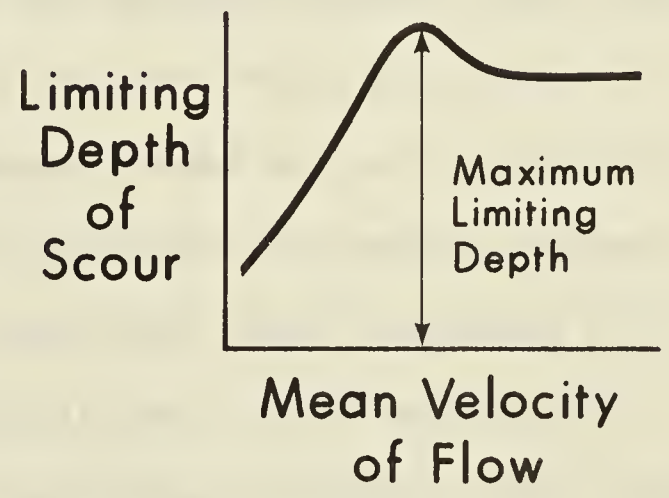


FIGURE 2-2(c) EFFECT OF VELOCITY OF FLOW ON LIMITING SCOUR DEPTH

FIGURE 2-2 MODEL RESULTS ON PIER SCOUR (NEILL, 1964)

of the approach flow as in Fig. 2-2(a). As is evident from the plot with decreasing length width ratio of a pier the angle of attack does not have much effect.

- (iv) Depth of Approach Flow:- Present data is rather inconclusive on this aspect. Laursen & Toch (1953) indicate a definite effect of depth of flow on scour depth, but Tison (1940), Chabert and Engeldinger (1956) and Neill (1964) report the opposite. The general consensus is that other conditions remaining the same, the depth of flow does not effect the scour depth (Neill, 1964).
- (v) Mean Velocity of the Approach Flow:- A typical plot of the scour depth against the mean velocity of the approach flow is Fig. 2-2(c). The depth of local scour increases with increasing mean velocity of the approach flow. This trend reaches a maximum just before the beginning of the general bed-transport in the approach channel, after which the scour depth drops off to a mean value about which it fluctuates with the transport in and out of the scour hole. Most of the test results agree with this general picture, only Varzeliots (1960) found scour depth increasing with increasing discharge intensity even after the start of general bed movement in the approach channel. Tison (1940) studied the effect of velocity distribution on the pier scour. His conclusion was: increasing the velocity gradient near the wall increases the scour depth.
- (vi) Variation of Pier Scour with Time:- In case of clear water scour the depth of scour varies asymptotically with time

as in Curve (a) of Fig. 2.2(b). In case of a sediment transporting flow it rapidly attains an equilibrium value, about which it fluctuates as in Curve (b) of Fig. 2-2(b).

- (vii) **Sediment Size:-** For the limited range (0.25 to 3mm) of sediments tested so far, the influence of sediment size on maximum scour depth is quite small (Neill, 1964). Qureshi (1965) found lesser maximum scour depth for light-weight materials than heavy-weight materials, under the same flow conditions. Laursen (1970) points out that the influence of sediment size is only present when the flow is non-transporting.

For scaling model to prototype the Froude number is used as the criteria for the dynamic similarity. With geometrical model to prototype scale ratios greater than 1/10, sand in model may represent sand in prototype. For scale ratios less than 1/10 it is doubtful if the model bed can represent the prototype bed.

2-5 Protection of Piers Against Local Scour

Pier protection efforts date as far back as 1893 (Engles, 1893) and 1903 (Spring, 1903) in the recorded history of hydraulics. Both of these references indicate the use of loose stone aprons as pier protection. Studies on other means of pier protection are of recent origin (Tison, 1940; Posey et al., 1951; Chabert and Engeldinger, 1956).

As a broad classification, there are three main methods of pier

protection against local scour. The first method consists of changing the shape of the pier to decrease the flow disturbance caused by it. The second method uses some auxiliary structures to reduce local scour at the pier. The third method uses such flexible protection as loose stone riprap to check local scour near the pier.

Methods Involving Change of Shape of the Pier

The commonest known method in this category is streamlining the face of the pier. Model test results of Tison (1940), Laursen and Toch (1953), Varzeliots (1960) bear ample evidence on the merits of streamlining the pier face. The reported merits of different streamlined pier shapes hold true as long as they are aligned parallel to the flow. Change in the direction of stream attack due to some channel activity or large sand dunes moving toward the pier can make the sharp-nosed pier act like a blunt-nosed pier.

Moore and Masch (1963) argued that if a pier is swept back at a proper angle near the bed, it can induce sufficient flow in the stream direction so as to cancel the downward flow produced by the velocity gradient in the approach flow. The effectiveness of such a swept backwards design would probably depend on the angle of sweep, its location, and the sharpness of the leading edge.

Knezevic (1960) and Tanka et al (1967) tried to check downward flow on the pier face by placing a cavity through the pier at the stream-bed. Knezevic (1960) found experimentally that on the average placing a slot reduced the maximum depth of scour to 46% from the corresponding value for the same pier without a cavity. Tanaka et al (1967) found that for piers of small diameters (3 cm) drilling a cavity through the pier had little effect on the maximum scour depth.

Shen, Schneider and Karaki (1966) roughened the upstream face of the pier in order to destroy the energy of the downward jet on the pier face. Their tests on such a pier did not show any promising results. They tried another test, with a roughened face pier supplemented by a roughened horizontal apron, without much success.

Knezevic (1960) experimented with exterior bands on the face of the pier to increase roughness and decrease the scour depth. By placing three bands, of width equal to $1/6$ of the pier width, the scour was reduced by about 30%.

Thomas (1967) reports extensive test with shields of different diameter placed on the pier at different heights as in Fig. 2-3(a). His general conclusions were:

- (a) Increasing the size of the shield as compared to the pier diameter decreases the maximum scour depth (Fig. 2-3(a)).
- (b) Location of the shield closer to the bed decreases the scour still further, Fig. 2-3(a) substantiates these conclusions. Moore and Masch (1963) suggest placing such a shield on the stream bed with a lip on the outer edge to deflect the secondary flows back towards the surface. For piers supported on piles, the pile cap can act as such a shield.

Schneible (1951) tested the effect of enlarged footings on pier scour. Fig. 2-3(b) shows some of these designs with their percentage reduction in scour depth. Chabert and Engeldinger (1956) found that enlarged footings, located about $1/2$ x pier diameter below the bed, and 2 to 6 times pier diameter, reduced the scour depth considerably.

Shen, Schneider and Karaki (1966) experimented with a cylinder split along its axis. Scour was reduced to some degree, but in actual

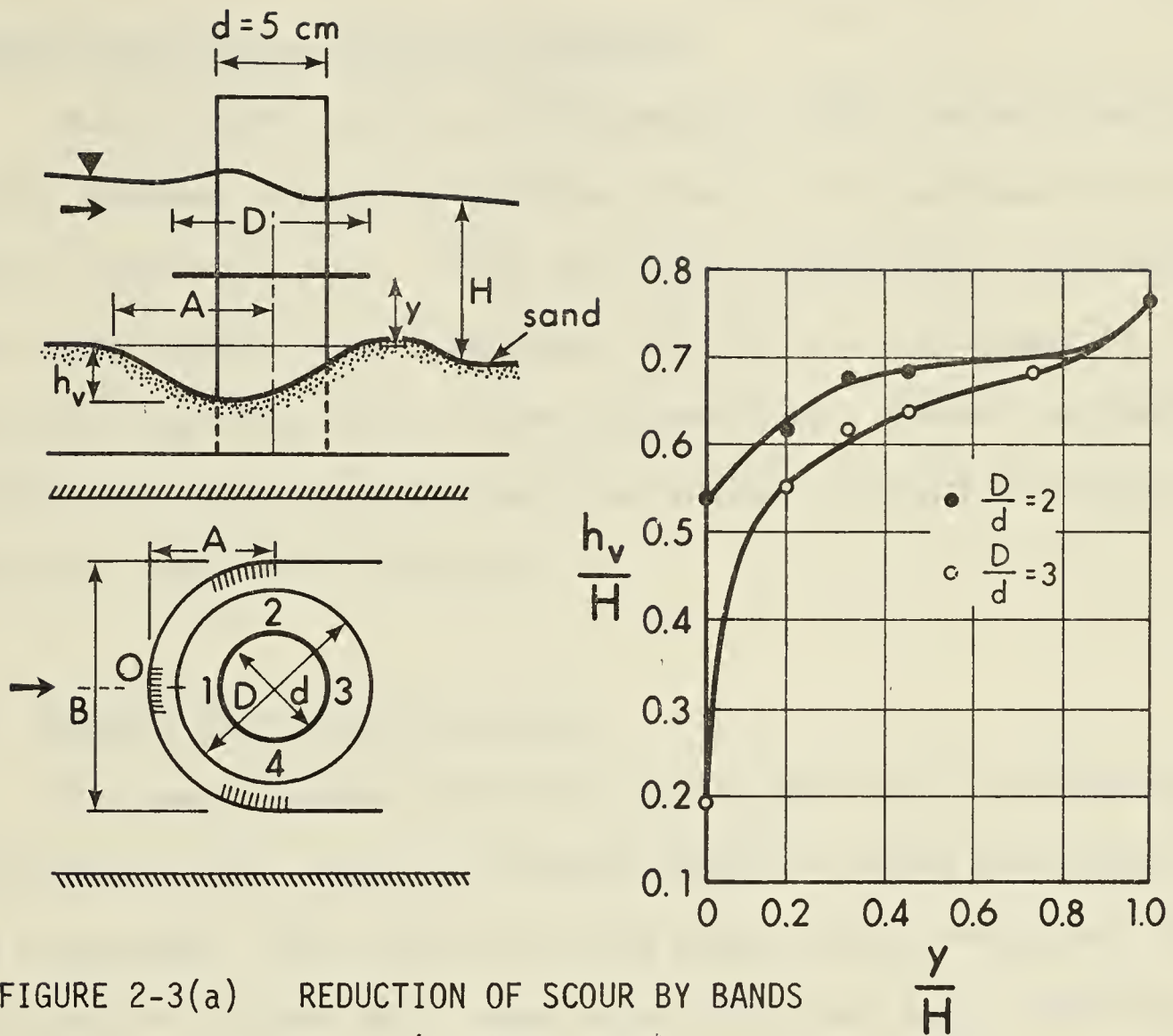


FIGURE 2-3(a) REDUCTION OF SCOUR BY BANDS
(THOMAS, 1967)

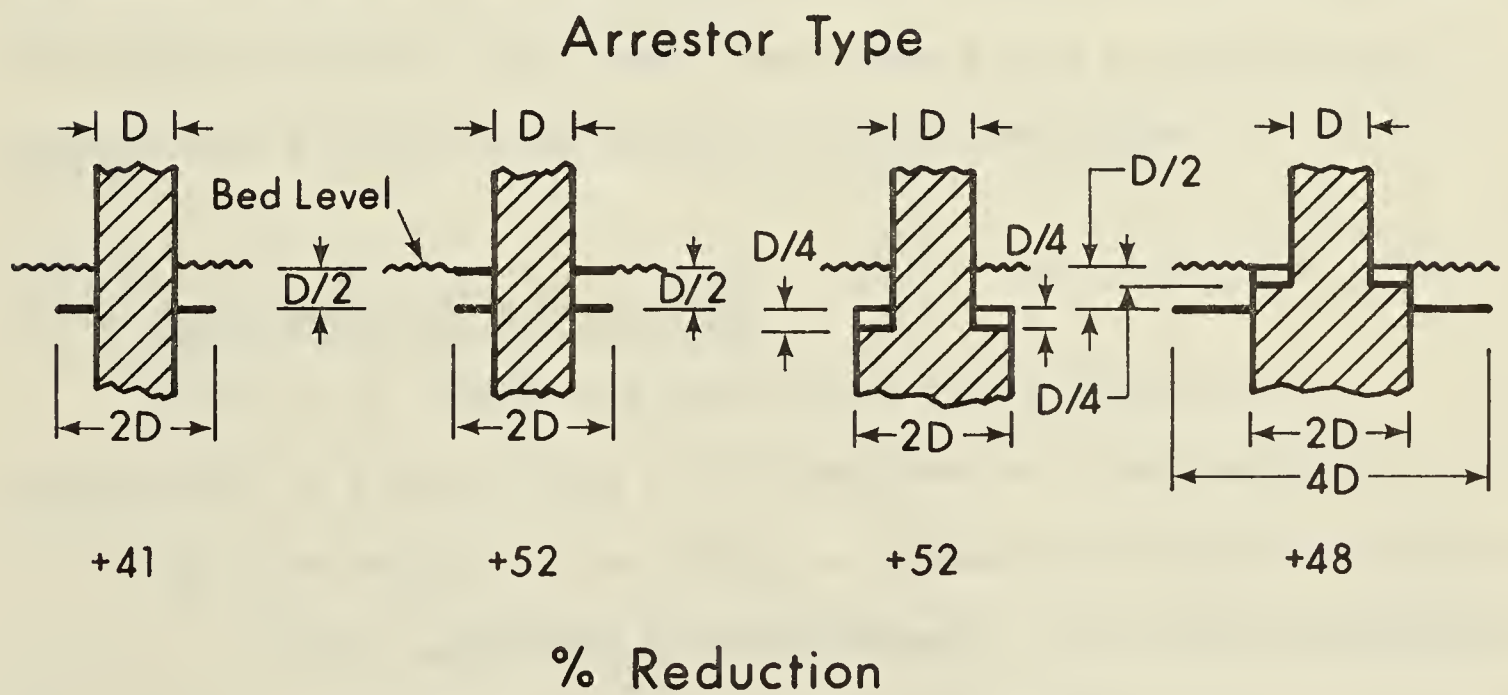


FIGURE 2-3(b) REDUCTION OF SCOUR BY ARRESTORS (SCHNEIBLE, 1950)

FIGURE 2-3 METHODS FOR REDUCING SCOUR AT PIERS

practice this advantage can be offset by debris clogged at the split.

Methods Requiring an Auxiliary Structure

Tison (1940), Chabert and Engledinger (1956) and Moore and Masch (1963) recommend placing an auxiliary pier or pile upstream of the main pier to reduce its scour. Moore and Masch recommend that this auxiliary pier could be quite short, submerged and its only requirement is to destroy the velocity gradient in the approach flow. Chabert and Engeldinger (1956) found experimentally that scour could be reduced by as much as 50% with piles upstream of the pier.

2-6 Flexible Protection for Piers

The name flexible protection implies any kind of protection, laid around a pier, which is flexible enough to follow the changes in the stream-bed. This could be a loose stone riprap, rubber mat, link chain mat or a screen mat. Posey et al (1951) and Appel (1950) did extensive studies on the qualitative merits of different types of mats for pier protection. Their model tests showed that a link chain mat underlain by a gravel layer provided the best protection.

2-6-1 Requirements of a Pier Riprap

Posey et al. (1951) and Sousa-Pinto (1959) describe the requirements of a pier riprap protection based on flume tests as:

- (a) The ability of the riprap to prevent the removal by suction of the underlying fine bed-material. The suction mechanism is attributed to the pressure differential between the nose of the pier and at points around the pier surface. This

pressure differential causes upflow through the protection (Posey, 1951). Posey et al (1951) point out the futility of using a rigid concrete mat as armouring around the pier. With the slight increase in approach velocity the thickness of the mat increases considerably and also it is difficult to make it water-tight at junctions with the pier.

- (b) The ability to bridge small discontinuities in the channel bed. A rigid mat would bridge the gaps like a rigid slab, thereby opening channels for currents to erode away the bed material. This characteristic is more important around the edges of the mat.
- (c) The ability of its material to withstand drag exerted by the increased velocities around the pier. These increased velocities can reach a magnitude of twice the free stream velocities around a cylindrical pier (Moore and Masch, 1963).

2-6-2 The Terzaghi-Vicksburg Riprap

The T-V riprap as introduced by Posey (1955), Manamperi (1952) and Sousa-Pinto (1959) refers to riprap graded in accordance with Terzaghi et al's (1948) criteria for graded filters as modified by the U. S. Waterways Experiment station at Vicksburgh (1948).

Tests at Iowa (Posey, 1957) showed that the use of uniformly sized protection for fine materials reached uneconomical proportions with the slight increase in velocity. Non-uniform grading of riprap protection was attempted by Manamperi (1952). The criteria adopted for proportioning the non-uniform protection was that of Terzaghi et al. (1948) for graded filters. Terzaghi et al. (1948) suggested that to prevent the

erosion of underlying materials and for its effective drainage, a filter grading should have the following relation to the base materials grading:

$$\frac{D_{15} \text{ Filter}}{D_{85} \text{ Base}} < 4$$

$$\frac{D_{15} \text{ Filter}}{D_{15} \text{ Base}} > 4$$

The Waterways Experiment Station at Vicksburg (1948) found that the above criteria is true only for uniformly graded materials. They proposed the modified criteria as:

$$\frac{D_{15} \text{ Filter}}{D_{85} \text{ Base}} < 5$$

$$4 < \frac{D_{15} \text{ Filter}}{D_{15} \text{ Base}} < 20$$

$$\frac{D_{50} \text{ Filter}}{D_{50} \text{ Base}} < 25$$

Tests by Manamperi (1952) showed that a 2" layer of T-V grading provided better protection than an 8" layer of uniform protection consisting of a uniform size equal to the largest of TV-grading. Shape of the riprap particles did not seem to have effect on the effectiveness of the protection (Posey, 1957).

2-6-3 Riprap Layer Thickness

Manamperi (1952) found experimentally that the thickness of a uniformly graded stone layer on a plane bed increased in more than direct proportion to the velocity at which movement of the sand-bed had commenced. From Manamperi's (1952) test data Sousa-Pinto (1959)

adopted the thickness of his pier riprap as 6.5 times the D_{50} of his riprap material.

Maza and Sanchez (1964) recommend the thickness of a riprap for a pier as the larger of the following two values:

- (a) width of the pier
- (b) three times the riprap diameter

ASCE (1948) and Waterways Experiment Station (1948) recommend the thickness of the riprap layer for slope protection as $1\frac{1}{2}$ times the average rock size or equal to the maximum rock size in the riprap.

Burgess and Hicks (1966), while investigating riprap protection under wave attack, found that:

- (a) Optimum stability is reached with a riprap layer of thickness $2.75 D_{50}$.
- (b) Below a thickness of $1.50 \times D_{50}$ the protection is unstable.

The California division of highways (1960) recommends a thickness of twice the largest stone size in the riprap for slope protection.

Inglis (1949) gives the thickness of the riprap for slope protection as:

$$T = 0.06 Q^{1/3}$$

Substituting in the above relation

$$Q = q \cdot B \quad B = 2.67 Q^{1/2}$$

the resulting equation is

$$T = 0.12 q^{2/3} \tag{2-11}$$

where: T = Thickness of the riprap (ft)

q = Discharge per unit width (Q/B)

B = Width of the channel (ft)

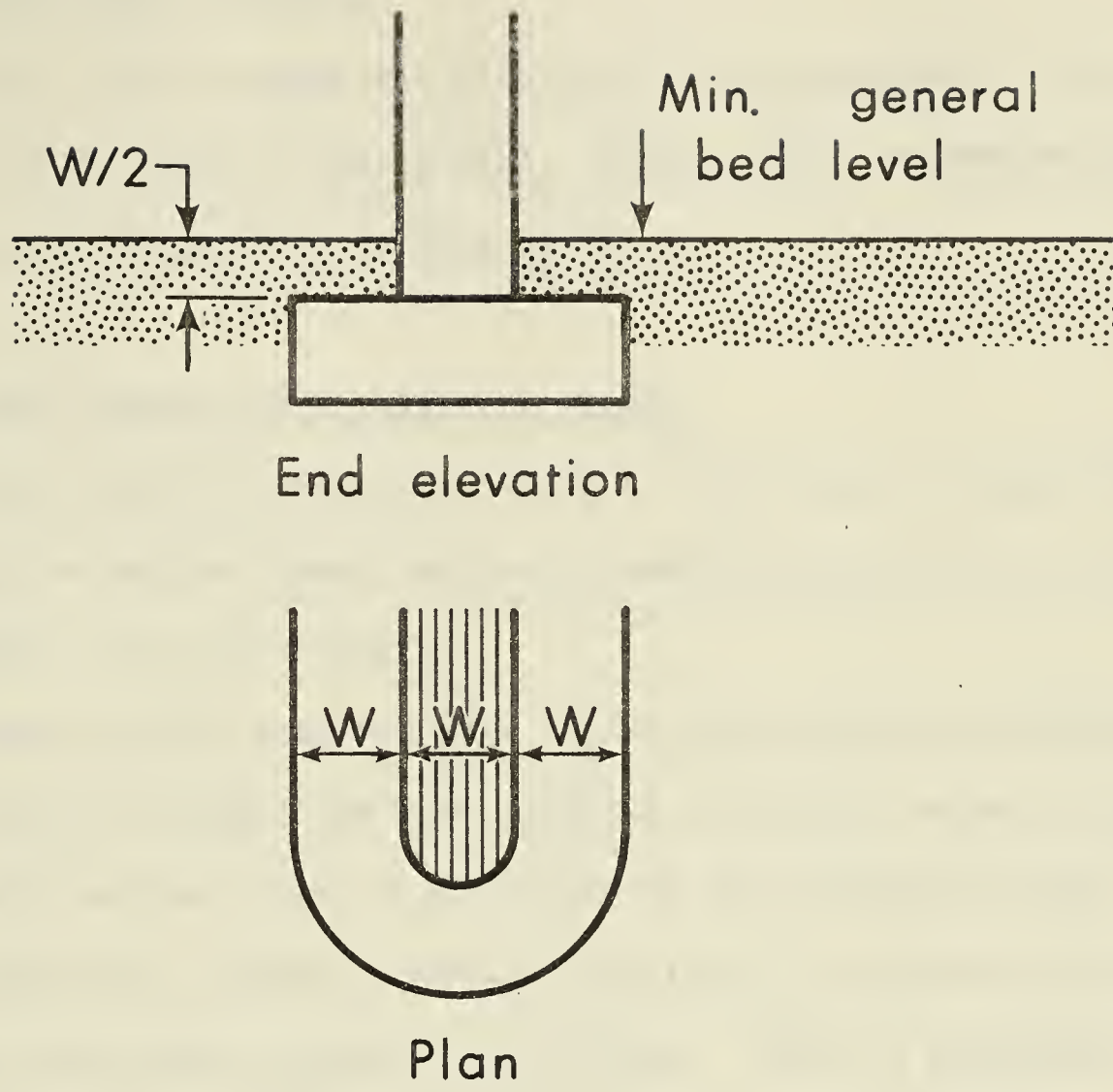


FIGURE 2-4(a) RIPRAP SHAPE (NEILL, 1964)

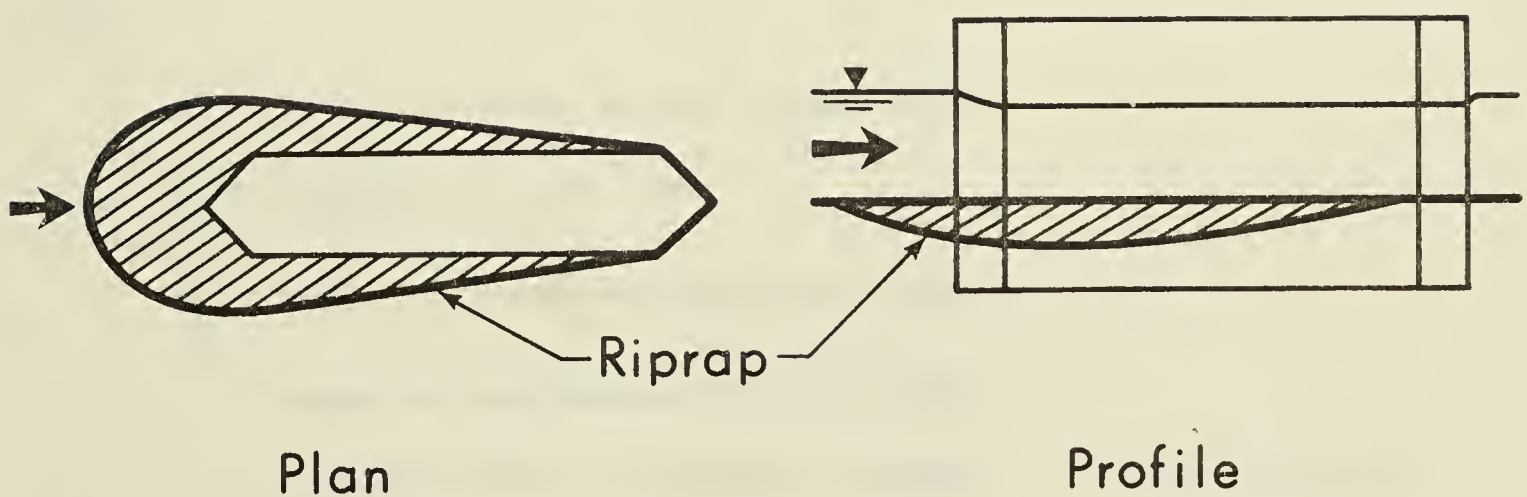


FIGURE 2-4(b) RIPRAP SHAPE (ENGLES, 1893)

FIGURE 2-4 DIFFERENT PLAN SHAPES FOR PIER RIPRAP

2-6-4 Plan Shape of Riprap

Neill (1964) suggests encircling the upstream half or the whole of the pier as shown in Fig. 2-4(a). Engles (1893) recommends placing riprap around the pier as in Fig. 2-4(b).

2-6-5 Areal Extent and Location of Riprap

Neill (1964) recommends placing of the riprap, at least half the pier width below the lowest bed level expected around the pier and of width equal to the pier width.

Engles (1893) suggests pier riprap flush with the streambed. Inglis (1949) considers the best location for riprap midway between high flood level and the level of the bottom of the foundation wells.

Sousa-Pinto (1959) experimentally tried to determine the extent of riprap protection around circular piers. From his experimental data, he recommended the following equation for pier riprap design:

$$\frac{a'}{d_p} = k \frac{d_s - p}{d_p}$$

where:

a' = lateral extent of the riprap layer

d_p = diameter of the pier

d_s = depth of scour without protection

p = depth of emplacement of the layer

k = experimentally determined constant = 1.8 for circular piers

The validity of the above equations was tested by ripraps of constant thickness laid at different depth and having different lateral extents. A typical arrangement of Sousa-Pinto's (1959) riprap is shown

in Fig. 2-5(a).

Sousa-Pinto (1959) tried to improve the performance of the pier riprap by tapering the edges as shown in Fig. 2-5(b). The rather qualitative result of this test indicated an improvement in performance of the riprap.

2-6-6 Significant Stone Size

Very little published information is available for selecting optimum stone size in a pier riprap. Blench (1961) has suggested 100 lb. size or three times the greatest stone that moves on the river bed during floods, whichever is the greatest. Varzeliots (1960) recommends using five times the size that moves on the river bed during similar conditions without the pier. Posey (1970) states that the size of such a rock is to be determined experimentally by trial and error.

Burgess and Hicks (1966) found that for a graded riprap slope protection under wave attack the D_{50} size is the significant size. Neill (1968) found similar results for initiation of motion of mixtures on plane beds.

Inglis, Thomas and Joglekar (1942) give an equation for the discharge that will cause failure at the nose of a ripraped pier as:

$$Q/b_1 = 2.3 g^{1/2} \left(\frac{W_*}{\gamma_s} \right)^{1/6} \left(\frac{\gamma_s}{\gamma} - 1 \right)^{1/2} b_1^{1/2} D^{2/3} D_p^{1/4} \quad (2-13)$$

where:

W_* = weight of the stone (lb)

b_1 = width of the pier (ft)

D_p = thickness of the protective layer (ft)

Table A-1 summarizes most of the design points for riprap by

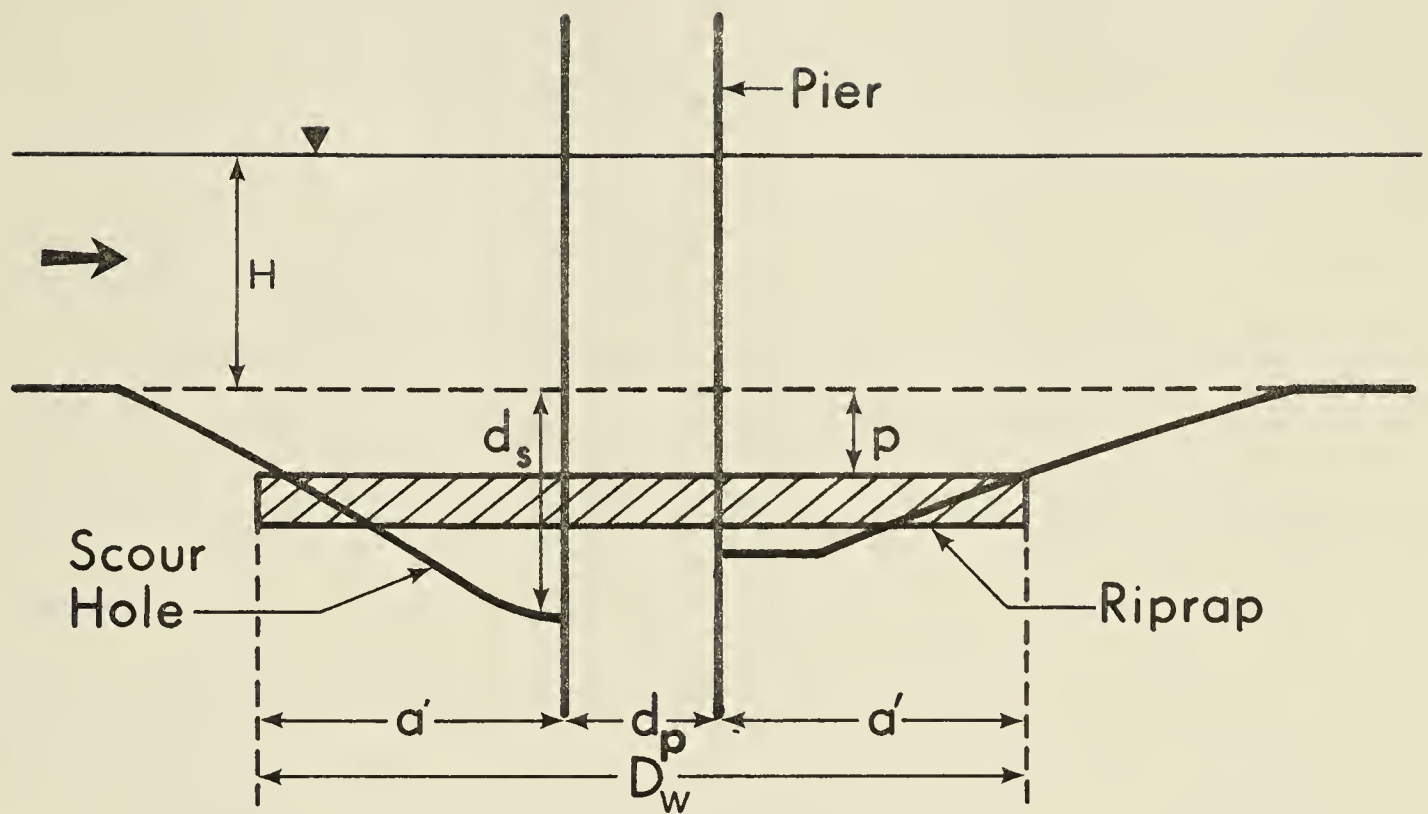


FIGURE 2-5(a) TYPICAL TEST RIPRAP

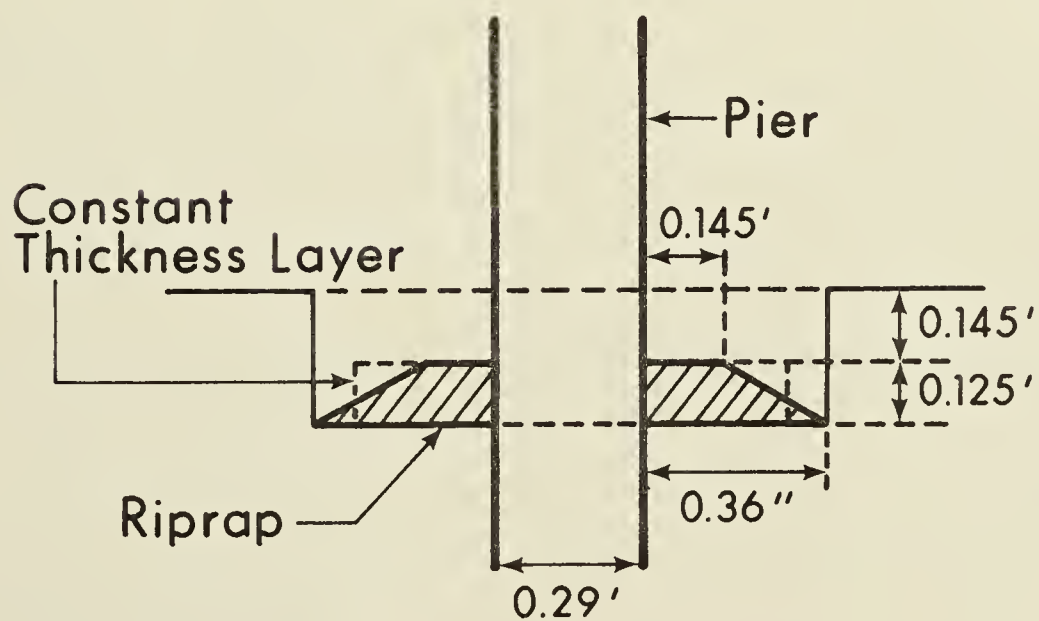


FIGURE 2-5(b) TAPERED RIPRAP

FIGURE 2-5 TEST PIER RIPRAP (SOUSA-PINTO, 1959)

different authors.

CHAPTER III

ANALYSIS OF THE PROBLEM

3-1 General

Physical laws governing any phenomenon of nature, can be expressed as mathematical relations between the parameters involved (Yalin, 1972). The success of any study on such complex phenomena as sediment transport, would depend on the complete statement of the problem in such a way that the mathematical laws can be formed (Blench, 1969).

The object of this chapter is to formulate such a statement for the case of the pier riprap situation. Proceeding from a complete statement of the problem, all the simplifying assumptions leading to the model experimentally tested are pointed out.

3-2 Pier Riprap Types

Different types of riprap around a pier behave differently under similar flow conditions. For this reason ripraps of different types are described here and their characteristics outlined.

In general there are two main types of pier ripraps:

(a) Launching type

(b) Flat riprap

(a) Launching Ripraps

In this riprap the stones are laid around the pier in such a way that the scour develops around the edges of the riprap. With the development of the scour hole, the stones are launched as shown in

Fig. 3-1(a). Obviously the stones in this riprap have to be heavy enough to withstand any movement due to the increased velocities and the fluctuating pressure around the pier. The quantity of the stones in the riprap should be sufficient to cover a slope of 1:2 to the maximum scour depth (Blench, 1969).

The action of this riprap will vary with its location relative to the stream-bed and the method of placement. Ripraps of this type have been placed level with stream-bed or dumped from the bridge in case of an emergency.

(b) Flat Riprap

This riprap is laid flat around the pier at or below the bed-level. The stone size in this riprap is designed to prevent any scour around the pier. The parameters that effect the behaviour of this type are:

1. Location of the riprap relative to the stream-bed.
2. Extent to which it covers the area around the pier.
3. Size of the under-lying material.

Fig. 3-1(b) and (c) illustrate the characteristics of a flat riprap. In Fig. 3-2

p = Location of the riprap relative to the stream-bed

a' = Uniform extent of the riprap around the pier

Presence of bed-load in the approach flow will probably make some difference on the behaviour of this type of riprap.

3-3 Riprap Studied

Fig. 3-3 is a sketch of the test riprap. Explanation of the various parameters and the riprap type is as follows:

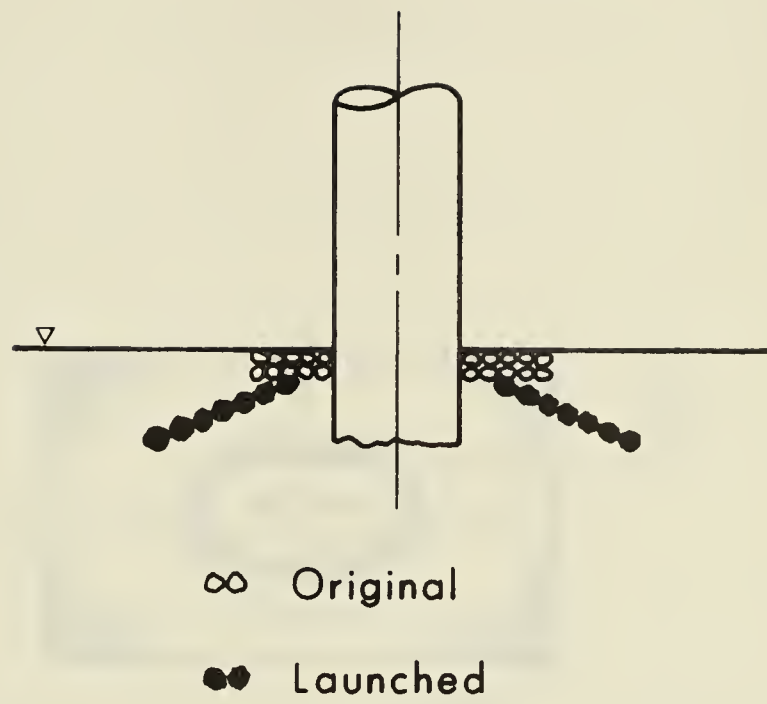


FIGURE 3-1(a) LAUNCHING RIPRAP

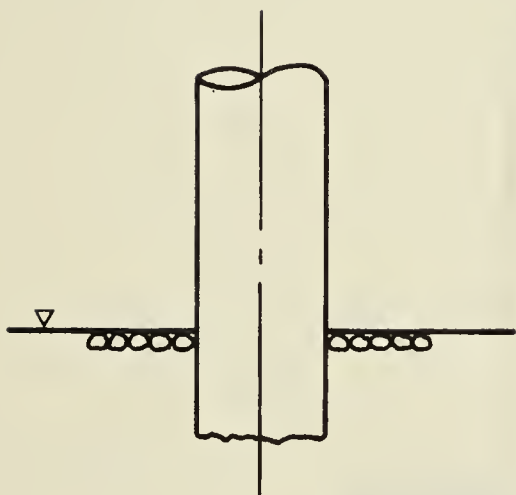
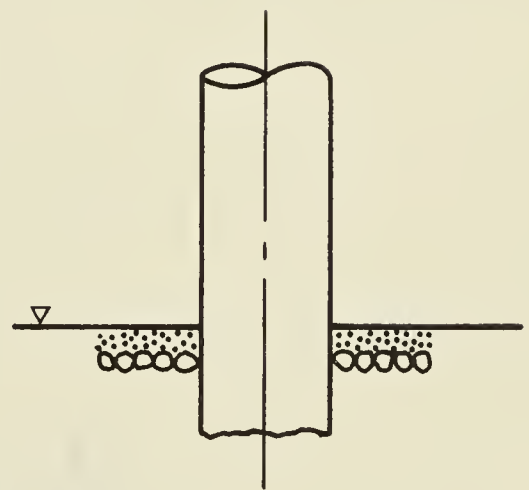
FIGURE 3-1(b)
FLAT RIPRAP AT BED LEVELFIGURE 3-1(c)
FLAT RIPRAP BELOW BED LEVEL

FIGURE 3-1 DIFFERENT TYPES OF PIER RIPRAP

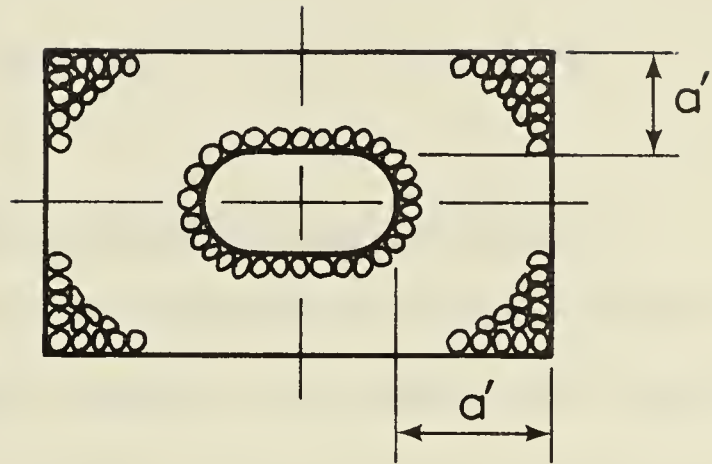


FIGURE 3-2(b) FLAT PIER RIPRAP IN PLAN

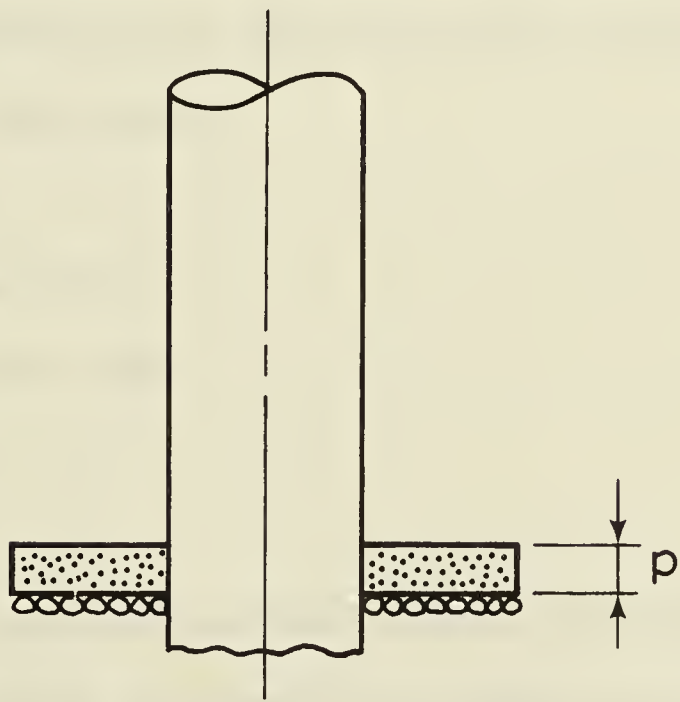


FIGURE 3-2(a) FLAT PIER RIPRAP IN ELEVATION

FIGURE 3-2 DESIGN PARAMETERS OF A FLAT PIER RIPRAP

1. Flat type.
2. $p = 0$ i.e. laid at the stream bed.
3. $a' = 2.5$ pier width
4. t = thickness of the approach bed

where:

t is the thickness of the riprap layer.

3-4 Characteristic Parameters of the Problem

The theory of dimensions describes a physical process by a set of independent elements. The number and type of these elements should be sufficient and necessary for completely defining the process. These elements are referred to as the "Characteristic Parameters". The characteristic parameters of the pier rip-rap situation can be described in terms of its components:

- (a) Pier
- (b) Approach bed
- (c) Fluid and flow
- (d) Riprap

(a) Pier

A pier is completely defined by its width (W) normal to the flow, shape factor (S_p) and length (L_p) along the flow. Roughness of the pier surface (E_p) does effect the boundary layer characteristics on its surface, but is neglected in the final analysis. Interaction of the pier and flow is represented by the angle (θ) the pier axis makes with the flow direction and the height (h_p) of the pier as compared to the depth of flow.

(b) Approach Bed

Significant parameters of the approach bed are its representative size (D), density of its particles (ρ_b) and the gradation parameter (δ_b) specifying the shape of its sieve curve. The shape of the approach bed particles is specified by X_b .

(c) Fluid and Flow

The approach fluid is defined by its mass density (ρ_f) and the kinematic viscosity (ν).

The flow is defined by the width (B) of the channel, the discharge (Q) through it and the uniform depth of flow (H) ahead of the pier.

The above phenomenon is taking place in earth's field of gravity, so g enters as one of the parameters. The approach flow is assumed to be without any bed-load.

(d) Riprap

The variables that define a riprap around a pier are:

1. Its lateral extent (a') as shown in Fig. 3-3.
2. Location of the top of the riprap relative to the approach bed (p) shown in Fig. 3-2.
3. Thickness (t) of the riprap layer.
4. Characteristics of the riprap material such as density (ρ_r), representative size (DG), gradation (δ_r) and the shape factor (X_r) of its particles.

(e) Characteristic Size (DG) and the Lateral Extent (a) of the Riprap

DG is defined as the riprap size that is just stable under a given set of pier and flow conditions. Uniformly sized materials were used for this study, so DG is the same as the measure for the materials i.e. their nominal size.

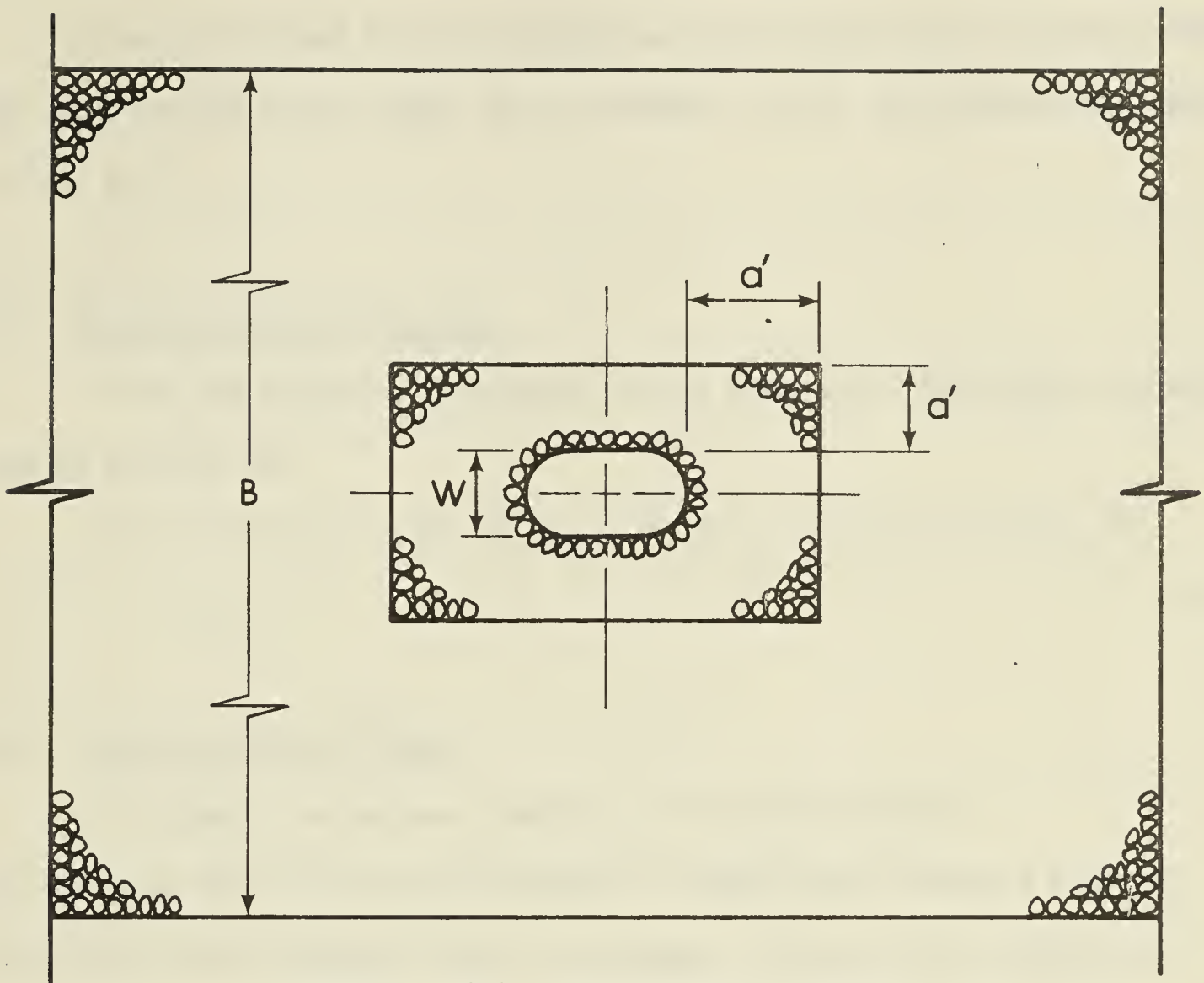


FIGURE 3-3(b) PLAN OF TEST RIPRAP

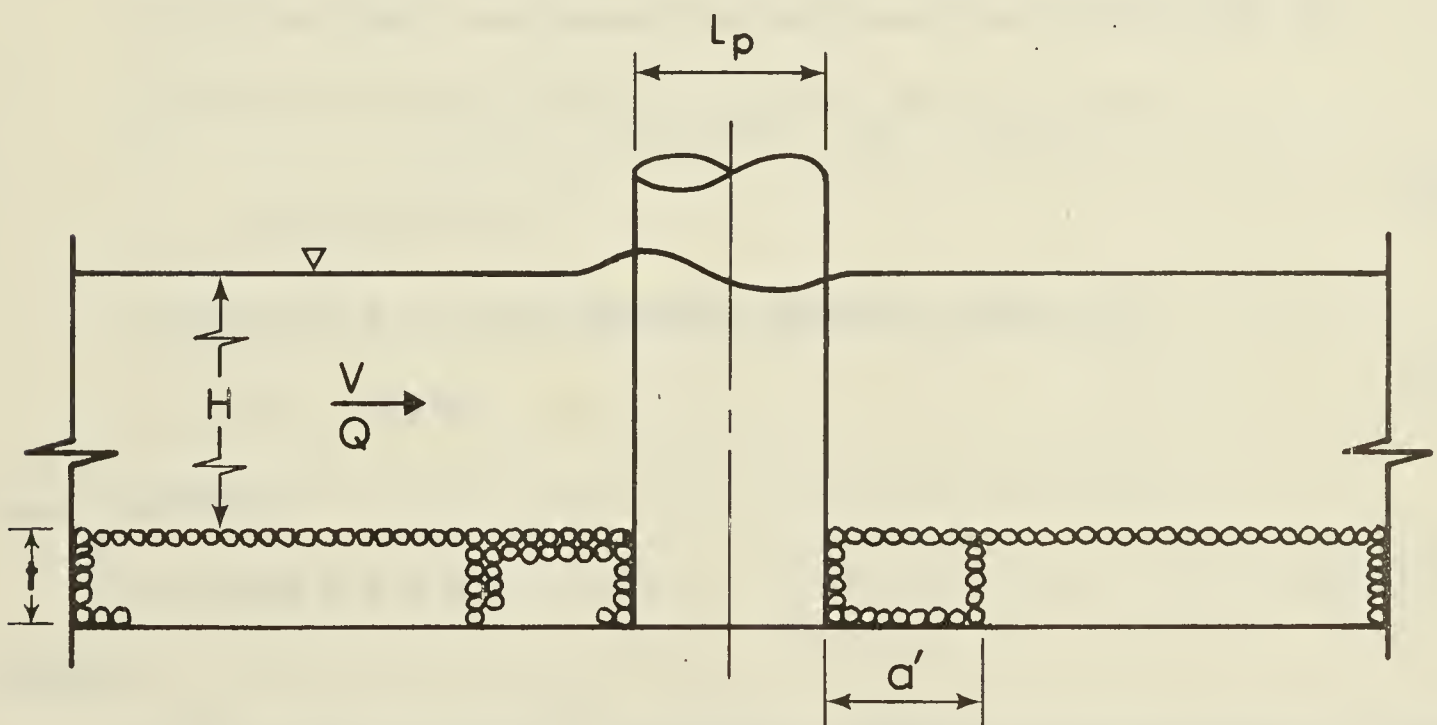


FIGURE 3-3(a) SECTION THROUGH TEST PIER

FIGURE 3-3 PLAN AND SECTION OF TEST RIPRAP

Lateral extent (a') is defined as the spread of the riprap around the pier beyond which there is no movement due to the disturbance caused by the pier.

3-5 The Functional Statement

From the previous statement of the problem, a functional relation can be written as:

$$0 = f(Q, B, H, \rho_f, v, W, S_p, L_p, E_p, \theta, h_p, D, \rho_b, \delta_b, X_b, a', P, t, DG, \rho_r, \delta_r, X_r, g) \quad (3-1)$$

3-6 The Experimental Model

The model tested was limited to the determination of the size (DG) for various grades of riprap laid around a pier of fixed size for a range of flow conditions. Other riprap parameters, (t), (p), and (a') were kept constant as explained in 3-3.

The functional statement for this case can be written as:

$$DG = f(Q, B, H, \rho_f, v, W, S_p, L_p, E_p, \theta, h_p, D, \rho_b, \delta_b, X_b, a', p, t, \rho_r, \delta_r, X_r, g) \quad (3-2)$$

Replacing g by the submerged specific weight γ'_s

$$\gamma'_s = (\rho_r - \rho_f)g \quad (3-3)$$

and Q by VMC

$$Q = VMC \times B \times H \quad (3-4)$$

where:

VMC = Mean velocity in approach channel at the instant of first instability.

With the above replacements Equation 3-2 is re-written as:

$$0 = f(DG, VMC, B, H, \rho_f, u, W, S_p, L_p, E_p, \theta, h_p, D, \rho_b, \delta_b, X_b, a', p, t, \rho_r, \delta_r, X_r, \gamma_s) \quad (3-5)$$

By standard techniques of dimensional analysis the above statement can be reduced to the non-dimensional form as:

$$0 = f(VMC DG/u, W/DG, H/DG, S_p, L_p/W, E_p, \theta, h_p/H, D/DG, B/H, \rho_b/\rho_f, \delta_b, X_b, a'/W, p/W, t/DG, \rho_r/\rho_f, \delta_r, X_r, \rho_f VMC^2/\gamma_s DG) \quad (3-6)$$

Analysis of the test data in these experiments is simplified by the constant value of many of the parameters. These parameters and their meaning are listed below:

Shape factor for the pier nose (S_p)

Parameter specifying the roughness of the pier surface (E_p)

Ratio of length of the pier to its width (L_p/W)

Angle of attack (θ)

Ratio of the pier height to the depth of flow ($h_p/H > 1$)

Ratio of approach bed grain size to riprap size (D/DG), and $B/H > 5$

Specific gravity (ρ_b/ρ_f)

Gradation parameter (δ_b) and shape parameter (X_b) for the approach bed.

Ratio of the lateral extent of the riprap to the pier width (a'/W)

Location of the riprap relative to stream bed (p/W)

Thickness to grain size ratio of the riprap (t/DG)

Specific gravity of the riprap material (ρ_r/ρ_f), their gradation (δ_r) and shape parameter (X_r)

The above-mentioned consideration reduces Equation 3.6 to:

$$f(VMC DG/u, \rho_f (VMC)^2/\gamma_s DG, H/DG, W/DG) = 0 \quad (3-7)$$

Other non-dimensional groups that can be evolved from the statement are:

$$f\left(\frac{VMC^2}{g(SG-1) DG}, \frac{\gamma'_S DG^3}{\rho_f v^2}, H/DG, W/DG\right) \quad (3-8)$$

$$f\left(\frac{q}{\sqrt{g(SG-1) DG^3}}, \gamma'_S DG^3/\rho_f v^2, H/DG, W/DG\right) = 0 \quad (3-9)$$

$$f\left(\frac{VMC}{\sqrt{g(SG-1) DG}}, \frac{\gamma'_S DG^3}{\rho_f v^2}, H/DG, W/DG\right) = 0 \quad (3-10)$$

CHAPTER IV

EXPERIMENTS

4-1 General

This chapter describes the experimental conditions, procedure for testing, the equipment and the materials tested. Criteria adopted for the failure of the riprap is also discussed.

4-2 Experimental Conditions and EquipmentFlume

Experiments were conducted in the Graduate Hydraulic Laboratory of the University of Alberta. A three foot wide flume with painted wooden sides and bottom was used. This flume is equipped with glass observation windows (4' x 2') at the test sections. Considerations for selecting a wide flume were:

- (i) If width to depth ratio of approach flow is 5 or more, the flow is essentially two-dimensional in the central portion of the channel.
- (ii) Width of the test pier should not create any constriction effects at the section.

This flume is equipped with a 10" supply line. Flow is measured by a magnetic flow meter located in the supply line. Discharge measurements are accurate within $\pm 1\%$ of full scale. Discharge to the flume is controlled by a butterfly valve located in the supply line.

Slope adjustment is provided by vertical levelling screws actuated by a motor and a gearing mechanism. Slope can be varied up

to 1%.

Tail gate control is provided by a motor and pulley system. Tail gate can be lowered or raised at a slow speed.

Approach Bed

Approach bed in the flume consisted of 54' length of gravel bed laid from 30' to 84' in the flume. The main consideration for the size selection of this material was to prevent any movement of the bed under the test conditions.

Test Piers

Test piers were standard round nosed piers as in Fig. 1-1 with 6" length.

Riprap Materials

The riprap materials tested were six sizes of closely sieved gravels, one size each of cellulose acetate and glass balls. All properties of the riprap materials are summarised in Table 4-1 and their shape is represented by Fig. 4-1.

Riprap Dimensions, Thickness and Shape

All test ripraps were laid in a square fashion around the pier as shown in Fig. 4-2. Uniform extent (a') around the pier was $2.5 \times$ pier width. The thickness of the riprap was made equal to the thickness of the approach bed.

Layout of the approach bed, riprap and the pier is given in Fig. 4-3.

Velocity Measurements

For each test the velocity profiles were taken at the first indication of instability. One profile was taken in mid-channel in the approach flow away from pier effects, and the other adjacent to the

TABLE 4-1
PHYSICAL PROPERTIES OF TEST RIPRAP MATERIALS

RIPRAP TYPE	PARTICLE SIZE			SPECIFIC GRAVITY	*NOMINAL DIA. mm	PARTICLE SHAPE						FALL VELOCITY cm/sec	DG ADOPTED SIZE mm	
	SIEVE LIMITS		*SIEVE DIA. mm			*	a mm	b mm	c mm	c/b	b/a			c/ $\sqrt{ab}$
	mm	mm												
1. GRAVEL	2.38	1.19	1.78	2.64	2.58							24.50	2.58	
2. GRAVEL	4.76	2.38	3.58	2.63	3.85							40.0	3.85	
3. GRAVEL	6.35	4.76	5.56	2.62	6.77	9.32	7.86	7.0	0.89	0.84	0.82	60.0	6.77	
4. GRAVEL	3/8"	6.35	7.93	2.64	10.50	14.2	12.6	10.3	0.82	0.89	0.77		10.50	
5. GRAVEL	1/2"	3/8"	11.11	2.65	14.40	20.7	18.2	14.6	0.80	0.88	0.75		14.40	
6. GRAVEL	Retained on 3/8"		9.51	2.64	13.60	19.6	16.5	14.0	0.85	0.84	0.78		13.60	
7. C.A. BALLS				1.31	6.39								6.40	
8. GLASS BALLS				2.29	15.70								15.70	

* a, b and c are the average measures for twenty-five particles of each materials.

* Sieve diameter is the arithmetic average of two nearest sieve sizes (one from which it is passing and the other on which it is retained).

* Nominal diameter is the diameter of a sphere of the same volume as the particle.

* a, b and c are the lengths of the longest, intermediate and the shortest mutually perpendicular axes of the particle.

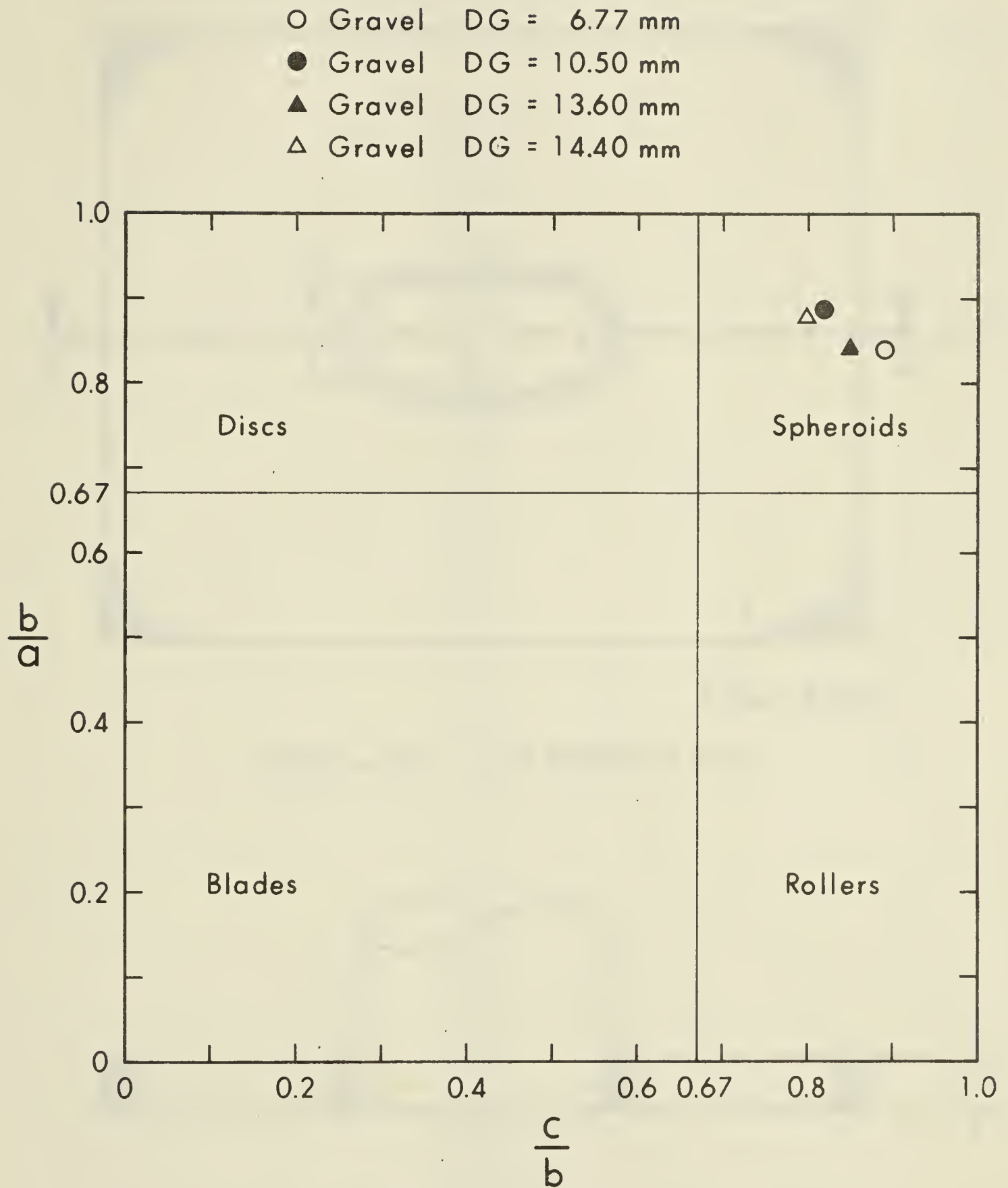


FIGURE 4-1 ZINGG'S DIAGRAM FOR SHAPE OF TEST GRAVELS

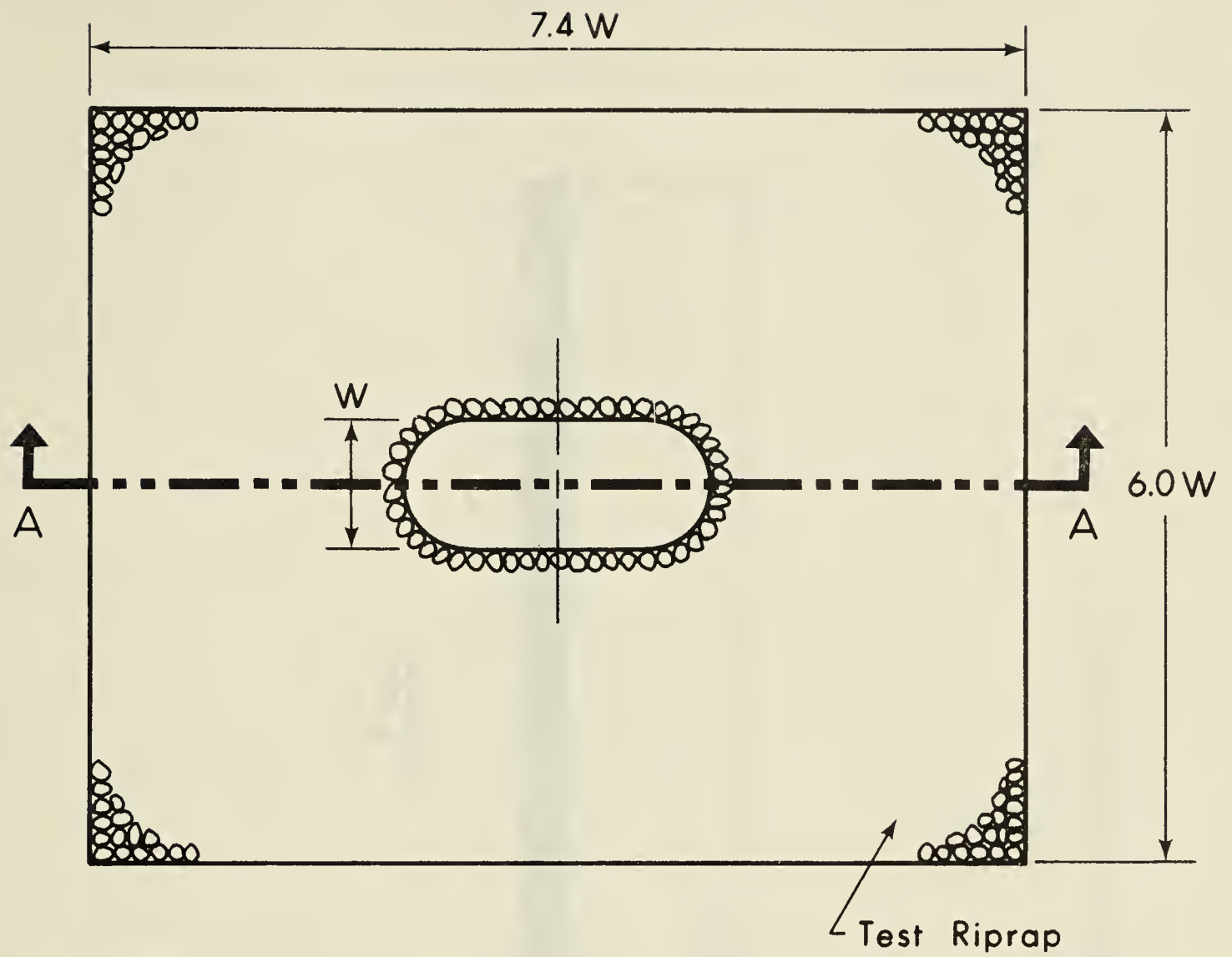
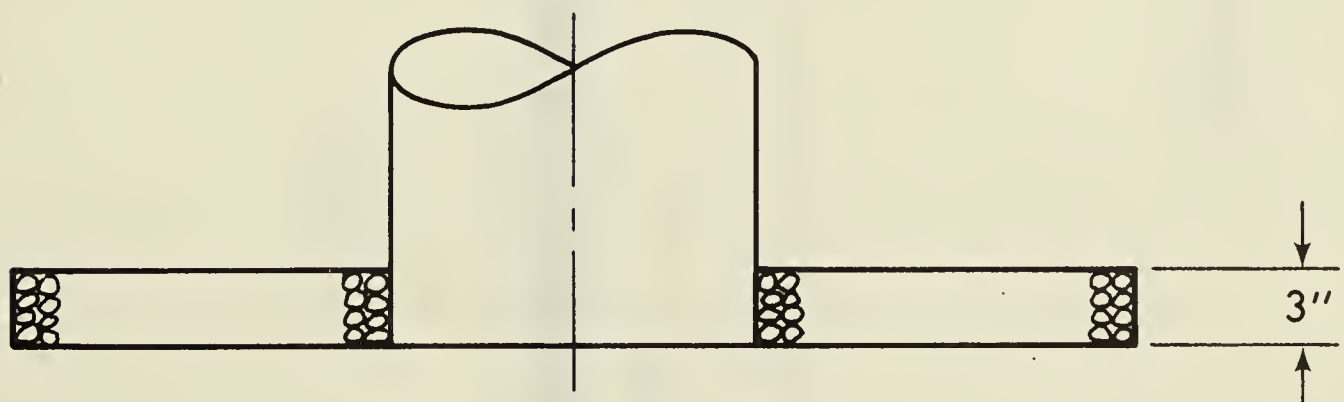


FIGURE 4-2(a) TEST RIPRAP IN PLAN



Section A - A

FIGURE 4-2(b) SECTION A-A

FIGURE 4-2 LAYOUT OF THE TEST PIER AND RIPRAP

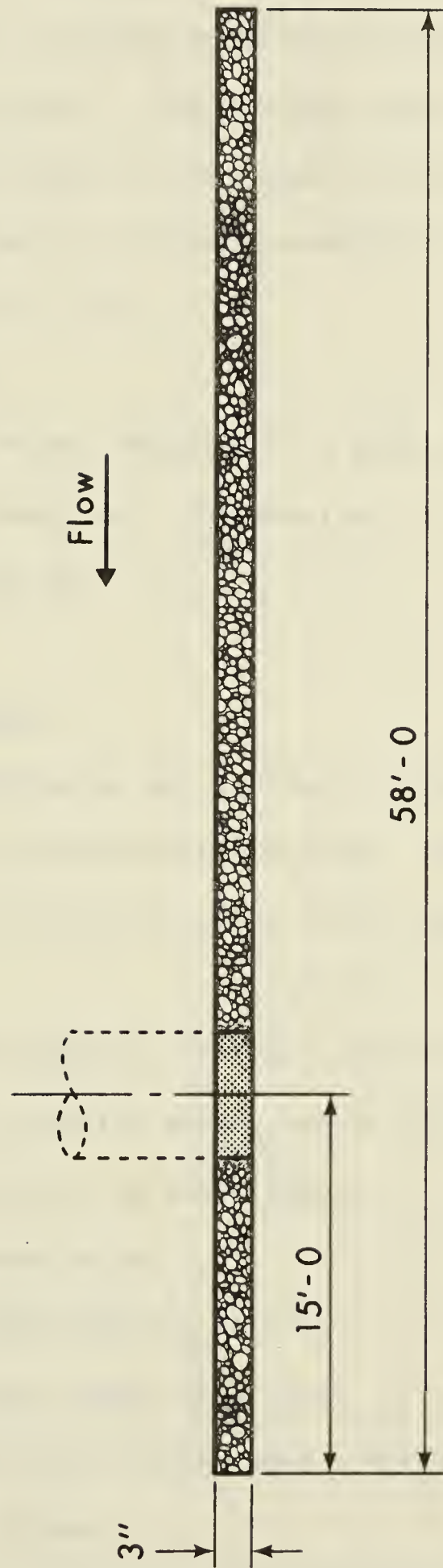


FIGURE 4-3 LAYOUT OF THE TEST SET UP

widest section of the pier. These two locations are shown in Fig. 4-4(a).

All velocity measurements were made with a miniature propeller type currentmeter. Propeller size for this currentmeter was 10 mm. Graphical record of velocity profiles was provided by connecting a currentmeter to a X-Y recorder. The recorder used in this study was model 2D-2A by Moseley and Company of Pasadena, California.

Typical velocity profiles recorded in mid-channel and at the pier face are shown in Fig. 4-5.

Depth Measurements

Depth of flow was measured by a point gauge with zero datum adjusted to the approach bed. The accuracy of depth measurement by this point gauge is ± 0.01 ft.

4-3 Failure Criteria

Before embarking on any testing programme it was necessary to define a failure criterion for the riprap. Possible choice lay between the displacement of the first stone to the complete destruction of the riprap.

Yalin (1972) suggests that for similarity of initial movement of beds with grains of similar shape, but of different sizes on a plane bed, the following conditions be satisfied:

- area of observation $\propto D^2$
- time of observation $t_o \propto D/V_*$
- observe same number of grains

Burghess and Hicks (1966) used a criteria based on the percentage damage to the riprap.

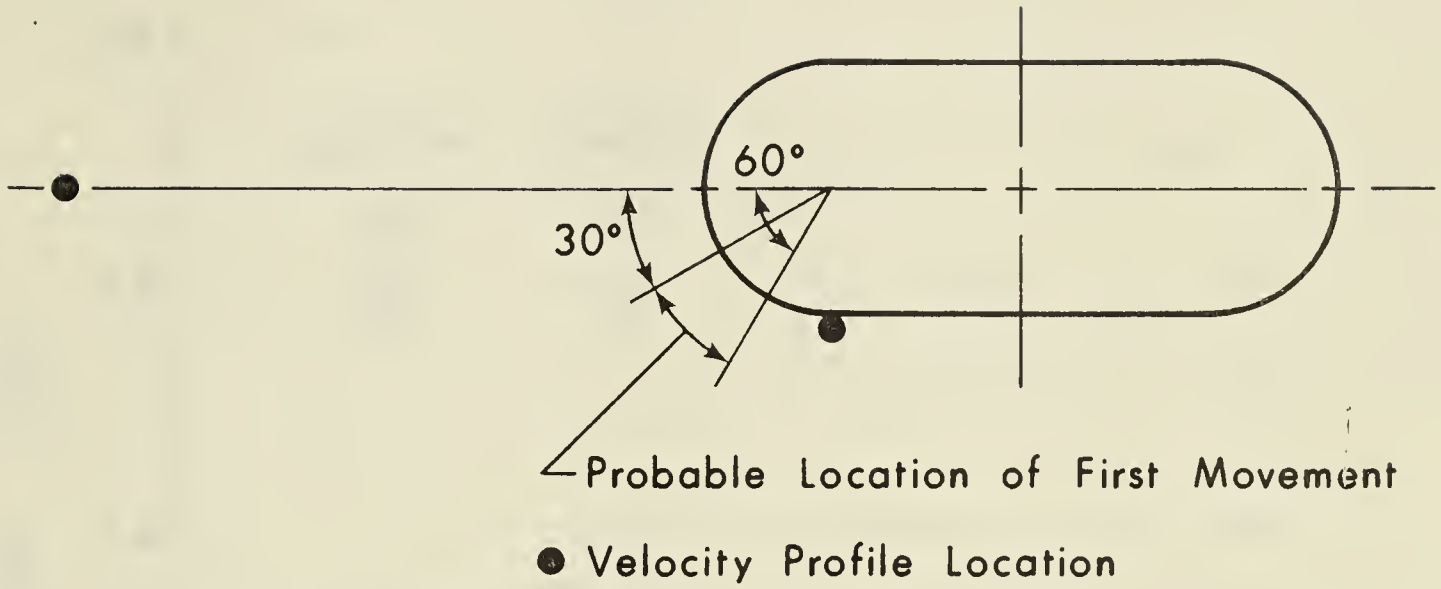


FIGURE 4-4(a)
LOCATION OF VELOCITY MEASUREMENT AND AREAS OF FIRST MOVEMENT

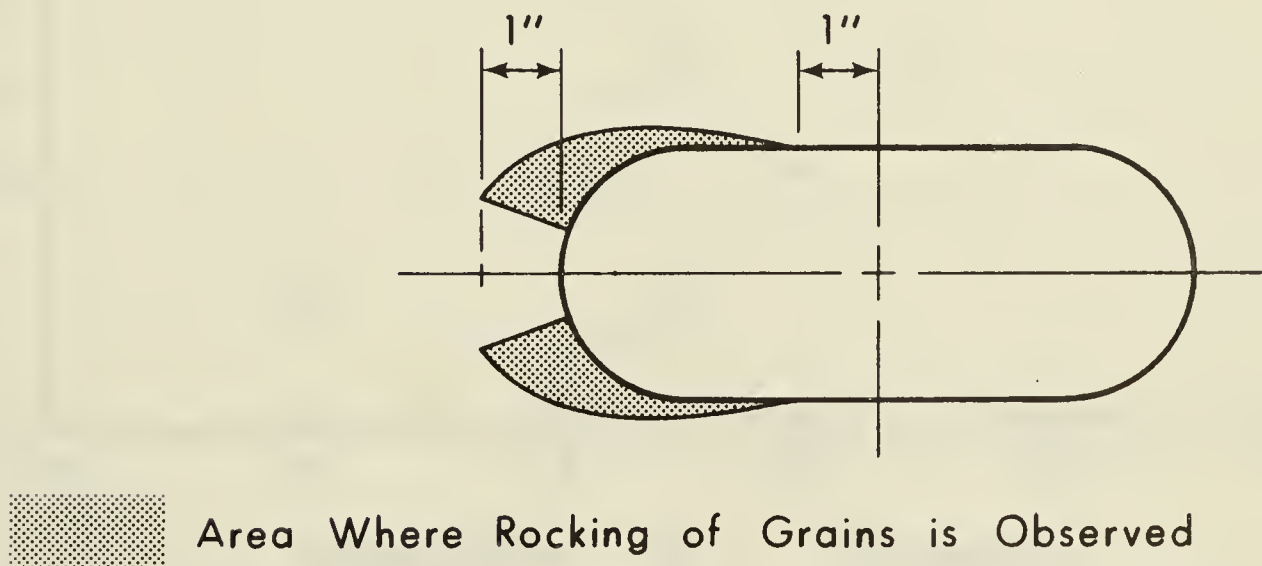


FIGURE 4-4(b) GENERAL AREAS OF INSTABILITY

FIGURE 4-4 OBSERVED AREAS OF INSTABILITY ON A PIER RIPRAP



FIGURE 4-5 TYPICAL OBSERVED VELOCITY PROFILES

Actual observation on test riprap around a pier showed that:

- (i) movement of first stone or stones took place at almost the same place.
- (ii) Once the first stone or stones were moved the failure of the riprap proceeded on quickly.

The above observations and the fact that the first movement always occurred close to the pier face, lead to the adoption of the following criteria for the riprap failure:

'A test riprap was deemed to have failed when any grain or grains are displaced from the front half of the riprap close to the pier face.'

For $W/DG < 4$ and $FR > 0.6$ relative riprap size became critical and the failure did not occur as per the above criteria. For stone size > 15 mm ($W/DG < 4$ and $FR \geq 0.6$) the separated boundary layer formed on the pier side became highly turbulent just downstream of the point of separation. With the increase in Froude number the point of separation of this boundary layer moved from the downstream end of the pier towards the upstream end. The high degree of turbulence in the separated layer combined with the small operating depths ($H < 20$ cm) caused the riprap to fail in the region downstream of the pier centre line while upstream of the centre line the riprap was still stable.

4-4 Test Procedure

The test procedure, in general, consisted of the following steps:

- (i) Draining the bed from the previous test.
- (ii) Laying the apron of the riprap material in the manner and shape as explained earlier.

- (iii) The surface of the riprap around the pier was leveled with the approach bed.
- (iv) The tail gate was raised and the desired discharge was set in the flume.
- (v) After establishing the desired discharge, the tail gate was lowered slowly in steps. At each step the slope in the flume was adjusted to provide a uniform depth of flow. This procedure was continued until the desired depth was obtained. At each step sufficient time was allowed for the conditions to stabilize. The area around the pier was observed for any displacement of riprap grains at all these steps.
- (vi) On the onset of riprap failure, vertical velocity profiles at mid-channel and close to the pier were taken.
- (vii) After the onset of first instability, sufficient time was allowed for the scour to develop.
- (viii) To make sure that the critical stage was reached, depth was further decreased thereby increasing the velocity. Rapid failure of the riprap confirmed conditions in excess of the critical stage.

4-5 Observation of Riprap Behaviour

In the beginning of the test programme, some tests were conducted to observe the behaviour of a pier riprap and how it failed. These observations were the basis for the formulation of the failure criteria described previously.

With velocities approaching the critical stage, grains in the

area marked in Fig. 4-4(b) started rocking. With the further increase in velocity, displacement of riprap grains usually occurred at locations marked in Fig. 4-4(b). The whole process of displacement and rocking of grains seemed to occur in bursts. For finer riprap materials, the grains are heaped up at the sides of the pier as shown in Fig. 4-6. Further increase in velocity causes the failure of the riprap in the front part of the pier. The final stabilized shape of the failed riprap is as shown in Fig. 4-6.

4-6 The Test Data

All the test data are listed in Tables 4-2, 4-3 and 4-4. Series A is the data for gravel ripraps around a 2.5 inch round-nosed pier, which was aligned parallel to the flow.

Series B is the data under essentially the same conditions but using glass balls and C. A. balls as the riprap materials. Series C is the data for various angles of attack and various pier sizes. In the Series C gravel of size 3.85 mm was used as the riprap material in all the tests.

In the tables 4-2 and 4-3 the pier Reynolds number (RN) has been computed to give some idea of the boundary layer separation from the pier sides.

Temperature of water in the flume was $66 \pm 1^{\circ}\text{F}$ in all the tests.

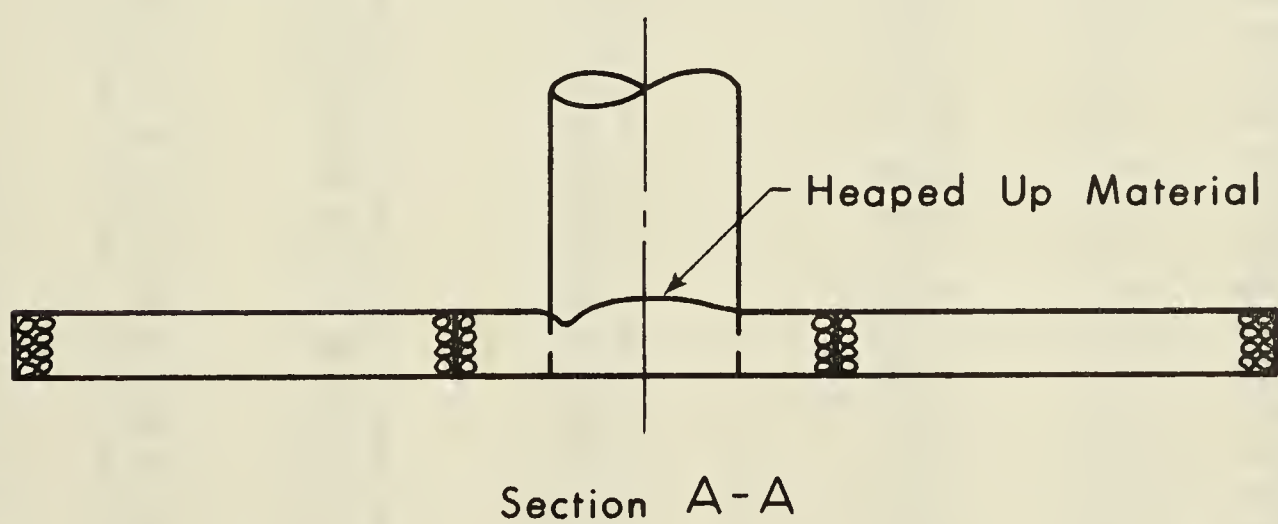
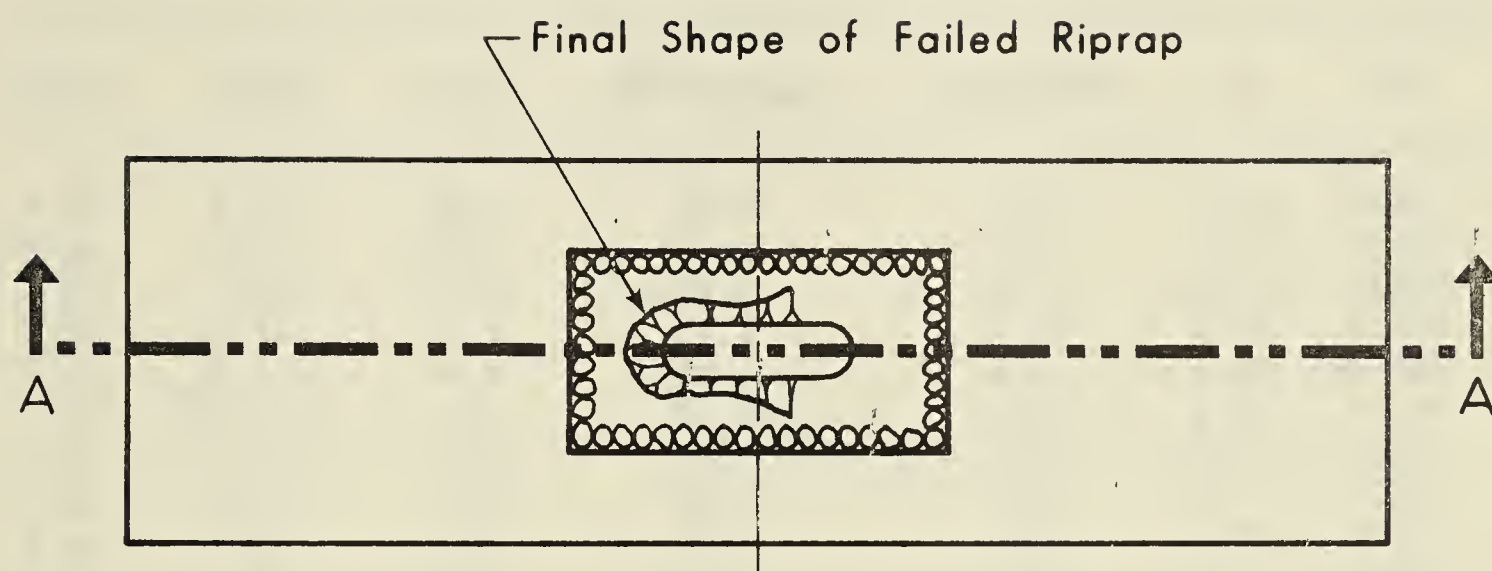


FIGURE 4-6 BEHAVIOUR OF SMALL SIZE ($<4\text{mm}$) MATERIAL AS PIER RIPRAP

TABLE 4-2

OBSERVED EXPERIMENTAL DATA AT RIPRAP FAILURE
SERIES A

DG (mm)	Q (cfs)	H (cm)	VMC (cm/sec)	S (PERCENT)	FR	RN
2.58	1.75	18.3	37.8	0.13	0.28	23447
2.58	1.05	12.2	31.1	0.05	0.28	19287
2.58	1.25	14.3	27.4	0.14	0.23	17018
2.58	0.77	7.3	27.1	0.12	0.32	16829
2.58	0.92	9.7	28.0	0.05	0.29	17396
3.85	0.60	4.2	54.9	0.32	0.85	34036
3.85	0.70	6.4	41.1	0.25	0.52	25527
3.85	0.82	7.9	39.0	0.22	0.44	24203
3.85	1.00	9.1	42.7	0.22	0.45	26472
3.85	1.20	10.6	41.7	0.24	0.41	25905
3.85	1.31	10.3	41.7	0.17	0.41	25905
3.85	1.49	14.0	39.0	0.16	0.33	24203
3.85	1.82	15.2	45.1	0.15	0.37	27985
3.85	1.90	16.1	45.7	0.17	0.36	28363
3.85	2.00	17.9	45.7	0.17	0.34	28363
3.85	2.30	20.1	42.7	0.15	0.30	26472
6.77	0.77	6.4	46.0	0.30	0.58	28552
6.77	0.96	7.3	50.6	0.31	0.60	31389
6.77	1.08	8.5	45.7	0.26	0.50	28363
6.77	1.40	10.9	47.5	0.22	0.46	29498
6.77	1.90	13.4	53.9	0.28	0.47	33469
6.77	2.05	16.4	52.7	0.26	0.41	32712
6.77	2.30	15.8	49.7	0.21	0.40	30821
6.77	2.71	17.9	45.7	0.22	0.34	28363
6.77	2.95	19.2	57.6	0.22	0.42	35738
10.50	1.40	9.7	55.5	0.31	0.57	34414
10.50	1.10	8.2	54.2	0.32	0.60	33658
10.50	1.72	10.6	57.0	0.30	0.56	35360
10.50	2.10	12.2	62.5	0.32	0.57	38763
10.50	2.50	14.6	66.7	0.28	0.56	41410
10.50	2.45	16.4	54.5	0.24	0.43	33847
10.50	3.20	18.9	62.2	0.22	0.46	38574
13.60	2.60	11.8	70.7	0.36	0.65	43869
13.60	2.95	14.0	71.3	0.32	0.61	44247
13.60	3.46	17.3	63.4	0.30	0.49	39330
13.60	3.85	19.5	62.2	0.26	0.45	38574
14.40	1.80	9.7	57.0	0.38	0.58	35360
14.40	2.00	10.6	57.9	0.34	0.57	35927
14.40	2.25	12.5	63.1	0.34	0.57	39141
14.40	2.60	15.8	68.3	0.28	0.55	42356

TABLE 4-2 (CONTINUED)

DG(mm)	Q(cfs)	H(cm)	VMC(cm/sec)	S(PERCENT)	FR	RN
14.40	3.10	16.7	69.5	0.26	0.54	43112
14.40	4.25	19.2	78.9	0.29	0.58	48974
15.90	4.75	19.5	77.7	0.31	0.56	48218

PIER SIZE = 2.5 in. RIPRAP SIZE = DG(mm) RIPRAP TYPE (NATURAL GRAVEL)
 THETA = 0° SPECIFIC GRAVITY = 2.64 T = 66°F
 FR = FROUDE NUMBER RN = PIER REYNOLD NUMBER

TABLE 4-3

OBSERVED EXPERIMENTAL DATA AT RIPRAP FAILURE
SERIES B

DG (mm)	Q (cfs)	H (cm)	VMC (cm/sec)	S (PERCENT)	FR	RN
15.70	2.55	12.8	77.7	0.32	0.69	48218
15.70	3.00	14.3	82.9	0.30	0.70	5.432
15.70	3.25	18.3	70.4	0.28	0.53	43680
15.70	4.15	18.6	80.5	0.28	0.60	49920

PIER SIZE = 2.5 in. RIPRAP SIZE = DG (mm) RIPRAP TYPE (GLASS BALLS)
 THETA = 0° SPECIFIC GRAVITY = 2.29 T = 66°F

DG (mm)	Q (cfs)	H (cm)	VMC (cm/sec)	S (PERCENT)	FR	RN
6.40	0.70	13.1	31.1	0.13	0.27	19287
6.40	0.82	16.4	32.0	0.18	0.25	19854
6.40	0.85	14.6	34.0	0.14	0.28	21178
6.40	0.90	17.3	30.0	0.16	0.23	18530

PIER SIZE = 2.5 in. RIPRAP SIZE = DG (mm) RIPRAP TYPE (C.A BALLS)
 THETA = 0° SPECIFIC GRAVITY = 1.31 T = 66°F

TABLE 4-4

OBSERVED EXPERIMENTAL DATA AT RIPRAP FAILURE
SERIES C

DG(mm)	Q(cfs)	H(cm)	VMC(cm/sec)	S(PERCENT)	FR	THETA(DEG)
3.85	0.85	9.1	35.3	0.18	0.37	15
3.85	1.25	11.5	34.4	0.19	0.32	15
3.85	1.70	17.6	30.8	0.17	0.23	15
3.85	1.85	19.2	32.9	0.17	0.24	15
3.85	1.45	15.5	33.2	0.17	0.27	15
3.85	1.00	11.5	25.0	0.16	0.23	25
3.85	1.25	13.7	26.8	0.16	0.23	25
3.85	1.85	18.9	33.8	0.17	0.25	25
3.85	1.50	16.4	28.3	0.15	0.22	25
3.85	0.85	8.8	28.0	0.16	0.30	25

PIER SIZE = 2.5 in. RIPRAP SIZE = DG(mm) RIPRAP TYPE (NATURAL GRAVEL)
 ANGLE OF ATTACK = 0° SPECIFIC GRAVITY = 2.64 T = 66°F

DG(mm)	Q(cfs)	H(cm)	VMC(cm/sec)	S(PERCENT)	FR	PIER SIZE(in.)
3.85	0.85	9.4	31.1	0.17	0.32	1.0
3.85	1.15	11.5	35.9	0.18	0.34	1.0
3.85	1.40	13.7	41.4	0.18	0.36	1.0
3.85	1.80	17.0	36.6	0.18	0.28	1.0
3.85	2.10	18.9	44.2	0.20	0.32	1.0
3.85	0.75	7.9	30.8	0.16	0.35	3.0
3.85	0.82	9.7	22.8	0.16	0.23	3.0
3.85	1.30	13.7	28.0	0.14	0.24	3.0
3.85	1.52	16.4	27.4	0.14	0.22	3.0
3.85	1.75	18.9	28.9	0.16	0.21	3.0

RIPRAP SIZE = DG(mm) THETA = 0° PIER SIZE = VARIABLE

CHAPTER V

PRESENTATION AND DISCUSSION OF RESULTS

5-1 General

Dimensional analysis in Chapter 3 was used to develop the functional equations:

$$f(VMC DG/v, \rho_f VMC^2/\gamma'_s DG, DG/H, W/DG) = 0 \quad (5-1)$$

$$f(q/\sqrt{g(SG-1)DG^3}, \gamma'_s DG^3/\rho_f v^2, H/DG, W/DG) = 0 \quad (5-2)$$

and

$$f(\frac{VMC}{\sqrt{g(SG-1)DG}}, \frac{\gamma'_s DG^3}{\rho_f v^2}, H/DG, W/DG) = 0 \quad (5-3)$$

Any one of these equations forms a mathematical model for the case of a pier riprap as stated in Chapter 3.

This chapter presents the evaluation and discussion of each of the above-mentioned equations based on the experimental data of Series A and B as listed in Table 4-2 and 4-3.

A comparative analysis is done with sediment transport data (Peterson, 1972) and initiation of motion data (Neill, 1967).

5-2 Definition of the Grain Size 'DG'

The size 'DG' used in the analysis of all the test data is defined as the diameter of a sphere of the same volume and specific gravity as the test material. In standard terminology (ASCE, 1962), this is referred to as the nominal diameter.

TABLE 5-1
NUMERICS COMPUTED FROM
DATA OF TABLE 4-2 and 4-3

DG (mm)	H (cm)	VMC*DG/v	FR1	DG/H	W/DG
2.58	7.3	684	1.77	0.04	24.6
2.58	9.7	707	1.89	0.03	24.6
2.58	12.2	784	2.33	0.02	24.6
2.58	14.3	692	1.81	0.02	24.6
2.58	18.3	953	3.44	0.01	24.6
3.85	4.2	2065	4.86	0.09	16.5
3.85	6.4	1549	2.73	0.06	16.5
3.85	7.9	1468	2.46	0.05	16.5
3.85	9.1	1606	2.94	0.04	16.5
3.85	10.6	1572	2.82	0.04	16.5
3.85	10.3	1572	2.82	0.04	16.5
3.85	14.0	1468	2.46	0.03	16.5
3.85	15.2	1698	3.29	0.03	16.5
3.85	16.1	1721	3.38	0.02	16.5
3.85	17.9	1721	3.38	0.02	16.5
3.85	20.1	1606	2.94	0.02	16.5
6.77	6.4	3047	1.95	0.11	9.4
6.77	7.3	3349	2.35	0.09	9.4
6.77	8.5	3026	1.92	0.08	9.4
6.77	10.9	3147	2.08	0.06	9.4
6.77	13.4	3571	2.67	0.05	9.4
6.77	16.4	3490	2.55	0.04	9.4
6.77	15.8	3289	2.27	0.04	9.4
6.77	17.9	3026	1.92	0.04	9.4
6.77	19.2	3813	3.05	0.04	9.4
10.50	9.7	5696	1.82	0.11	6.0
10.50	8.2	5570	1.74	0.13	6.0
10.50	10.6	5852	1.92	0.10	6.0
10.50	12.2	6415	2.31	0.09	6.0
10.50	14.6	6853	2.64	0.07	6.0
10.50	16.4	5602	1.76	0.06	6.0
10.50	18.9	6384	2.29	0.06	6.0
13.60	11.8	9404	2.29	0.11	4.7
13.60	14.0	9485	2.33	0.10	4.7
13.60	17.3	8431	1.84	0.08	4.7
13.60	19.5	8269	1.77	0.07	4.7
14.40	9.7	8026	1.40	0.15	4.4
14.40	10.6	8155	1.45	0.13	4.4
14.40	12.5	8884	1.72	0.12	4.4
14.40	15.8	9614	2.01	0.09	4.4
14.40	16.7	9786	2.09	0.09	4.4
14.40	19.2	11116	2.69	0.07	4.4
15.90	19.5	12084	2.36	0.08	4.0

TABLE 5-1 (CONTINUED)

DG (mm)	H (cm)	VMC*DG/ ν	FR1	DG/H	W/DG
15.70	12.8	11932	3.04	0.12	4.0
15.70	14.3	12728	3.46	0.11	4.0
15.70	18.3	10809	2.50	0.09	4.0
15.70	18.6	12354	3.26	0.08	4.0
6.40	13.1	1945	4.97	0.05	9.9
6.40	16.4	2003	5.26	0.04	9.9
6.40	14.6	2136	5.99	0.04	9.9
6.40	17.3	1869	4.59	0.04	9.9

ν IS KINEMATIC VISCOSITY

FR1 IS MOBILITY NUMBER = $\rho_f \text{ VMC}^2 / \gamma_s' \text{ DG}$

TABLE 5-2

NUMERICS COMPUTED FROM
DATA OF TABLE 4-2 and 4-3

DG (mm)	H (cm)	H/DG	FR'	DG/W
2.58	7.3	28.37	37.78	0.04
2.58	9.7	37.83	52.07	0.04
2.58	12.2	47.29	72.17	0.04
2.58	14.3	55.56	74.82	0.04
2.58	18.3	70.93	131.60	0.04
3.85	4.2	11.09	24.45	0.06
3.85	6.4	16.64	27.51	0.06
3.85	7.9	20.60	32.29	0.06
3.85	9.1	23.77	40.75	0.06
3.85	10.6	27.73	46.53	0.06
3.85	10.3	26.94	45.20	0.06
3.85	14.0	36.44	57.13	0.06
3.85	15.2	39.61	71.87	0.06
3.85	16.1	41.99	77.14	0.06
3.85	17.9	46.74	85.87	0.06
3.85	20.1	52.29	89.66	0.06
6.77	6.4	9.46	13.20	0.11
6.77	7.3	10.81	16.58	0.11
6.77	8.5	12.61	17.48	0.11
6.77	10.9	16.22	23.37	0.11
6.77	13.4	19.82	32.41	0.11
6.77	16.4	24.33	38.87	0.11
6.77	15.8	23.43	35.27	0.11
6.77	17.9	26.58	36.83	0.11
6.77	19.2	28.38	49.55	0.11
10.50	9.7	9.30	12.55	0.17
10.50	8.2	7.84	10.35	0.17
10.50	10.6	10.17	14.10	0.17
10.50	12.2	11.62	17.67	0.17
10.50	14.6	13.94	22.65	0.17
10.50	16.4	15.69	20.82	0.17
10.50	18.9	18.01	27.25	0.17
13.60	11.8	8.75	13.22	0.21
13.60	14.0	10.32	15.73	0.21
13.60	17.3	12.78	17.33	0.21
13.60	19.5	14.35	19.08	0.21
14.40	9.7	6.78	8.03	0.23
14.40	10.6	7.41	8.92	0.23
14.40	12.5	8.68	11.38	0.23
14.40	15.8	11.01	15.62	0.23
14.40	16.7	11.65	16.82	0.23
14.40	19.2	13.34	21.89	0.23
15.90	19.5	12.28	18.87	0.25

TABLE 5-2 (CONTINUED)

DG (mm)	H (cm)	H/DG	FR'	DG/W
15.70	12.8	8.16	14.23	0.25
15.70	14.3	9.13	16.98	0.25
15.70	18.3	11.66	18.41	0.25
15.70	18.6	11.85	21.39	0.25
6.40	13.1	20.49	45.67	0.10
6.40	16.4	25.73	59.04	0.10
6.40	14.6	22.87	55.98	0.10
6.40	17.3	27.16	58.17	0.10

$$FR' = \frac{q}{\sqrt{g(SG-1)DG^3}}$$

TABLE 5-3
NUMERICS COMPUTED FROM
DATA OF TABLE 4-2 and 4-3

DG (mm)	H (cm)	FRD	H/DG	DG/W
2.58	7.3	1.33	28.37	0.04
2.58	9.7	1.38	37.83	0.04
2.58	12.2	1.53	47.29	0.04
2.58	14.3	1.35	55.56	0.04
2.58	18.3	1.86	70.93	0.04
3.85	4.2	2.20	11.09	0.06
3.85	6.4	1.65	16.64	0.06
3.85	7.9	1.57	20.60	0.06
3.85	9.1	1.71	23.77	0.06
3.85	10.6	1.68	27.73	0.06
3.85	10.3	1.68	26.94	0.06
3.85	14.0	1.57	36.44	0.06
3.85	15.2	1.81	39.61	0.06
3.85	16.1	1.84	41.99	0.06
3.85	17.9	1.84	46.74	0.06
3.85	20.1	1.71	52.29	0.06
6.77	6.4	1.39	9.46	0.11
6.77	7.3	1.53	10.81	0.11
6.77	8.5	1.39	12.61	0.11
6.77	10.9	1.44	16.22	0.11
6.77	13.4	1.63	19.82	0.11
6.77	16.4	1.60	24.33	0.11
6.77	15.8	1.51	23.43	0.11
6.77	17.9	1.39	26.58	0.11
6.77	19.2	1.75	28.38	0.11
10.50	9.7	1.35	9.30	0.17
10.50	8.2	1.32	7.84	0.17
10.50	10.6	1.39	10.17	0.17
10.50	12.2	1.52	11.62	0.17
10.50	14.6	1.62	13.94	0.17
10.50	16.4	1.33	15.69	0.17
10.50	18.9	1.51	18.01	0.17
13.60	11.8	1.51	8.75	0.21
13.60	14.0	1.52	10.32	0.21
13.60	17.3	1.36	12.78	0.21
13.60	19.5	1.33	14.35	0.21
14.40	9.7	1.18	6.78	0.23
14.40	10.6	1.20	7.41	0.23
14.40	12.5	1.31	8.68	0.23
14.40	15.8	1.42	11.01	0.23
14.40	16.7	1.44	11.65	0.23
14.40	19.2	1.64	13.34	0.23

TABLE 5-3 (CONTINUED)

DG (mm)	H (cm)	FRD	H/DG	DG/W
15.90	19.5	1.54	12.28	0.25
15.70	12.8	1.74	8.16	0.25
15.70	14.3	1.86	9.13	0.25
15.70	18.3	1.58	11.66	0.25
15.70	18.6	1.81	11.85	0.25
6.40	13.1	2.23	20.49	0.10
6.40	16.4	2.29	25.73	0.10
6.40	14.6	2.45	22.87	0.10
6.40	17.3	2.14	27.16	0.10

$$\text{FRD} = \frac{\text{VMC}}{\sqrt{g(\text{SG}-1)\text{DG}}}$$

Table 4-1 lists some other measures of the size of the test materials. The adopted size 'DG' is quite close to the length of the shortest axis 'C' of the particle.

5-3 Plotting and Analysis

Plotting of the test data was carried out using the Auto-plotting Sub-routine GPL, available in the University of Alberta Computing Centre library programmes.

For determining the lines of best fit, a regression analysis was carried out using a linear model of the type:

$$Y = a + bX$$

where

Y is the dependent variable

X is the independent variable

a, b are the coefficients for the best fit line

For the exponential equations of the type $Y = a x^b$ the data was transformed to logarithms to the base 10 to conform to the above model.

5-4 Computation of Numerics from Experimental Data

All the non-dimensional groups in Equation 5-1, 5-2 and 5-3 were computed using the experimental data of series A and B. Tables 5-1, 5-2 and 5-3 list all these computed parameters.

In Table 5-1

$$FR1 = \rho_f VMC^2 / \gamma_s' DG$$

In Table 5-2

$$FR' = q / \sqrt{(SG-1)gDG^3}$$

Table 5-3

$$FRD = VMC / \sqrt{g(SG-1) DG}$$

The fourth parameter $\left(\frac{\gamma_s' DG^3}{\rho_f v^2} \right)$ in Equation 5-2 and 5-3

being constant for each material is marked on the plots.

5-5 Relationship Between VMC DG/v and FR1

The relationship of equation 5-1 is plotted in Fig. 5-1 as VMC DG/v vs FR1. The values of the third parameter DG/H are marked for each point and lines for equal W/DG are drawn through each set of points.

For the experimental data of this study v was nearly constant in all the tests, so for a particular material size the only variable in VMC DG/v and FR1 is VMC. In fact Figure 5-1 is a spurious correlation between VMC and VMC² with a slope 2:1 as indicated by lines representing different W/DG ratios.

5-6 Relationship between DG/H and FR1

An alternative relationship to Equation 5-1 is

$$f(\rho_f VMC^2 / \gamma_s' DG, \gamma_s' DG^3 / \rho_f v^2, DG/H, W/DG) = 0 \quad (5-4)$$

The numerical values for Equation 5-4 are plotted in Fig. 5-2.

In Equation 5-4 the grain Reynold's number (VMC DG/v) present in Equation 5-1 has been replaced by $\gamma_s' DG^3 / \rho_f v^2$. For a given material and pier size $\gamma_s' DG^3 / \rho_f v^2$ and W/DG are constants.

On Fig. 5-2 lines of average trend for each set of points can be drawn by eye. The overall scatter and the scatter around each

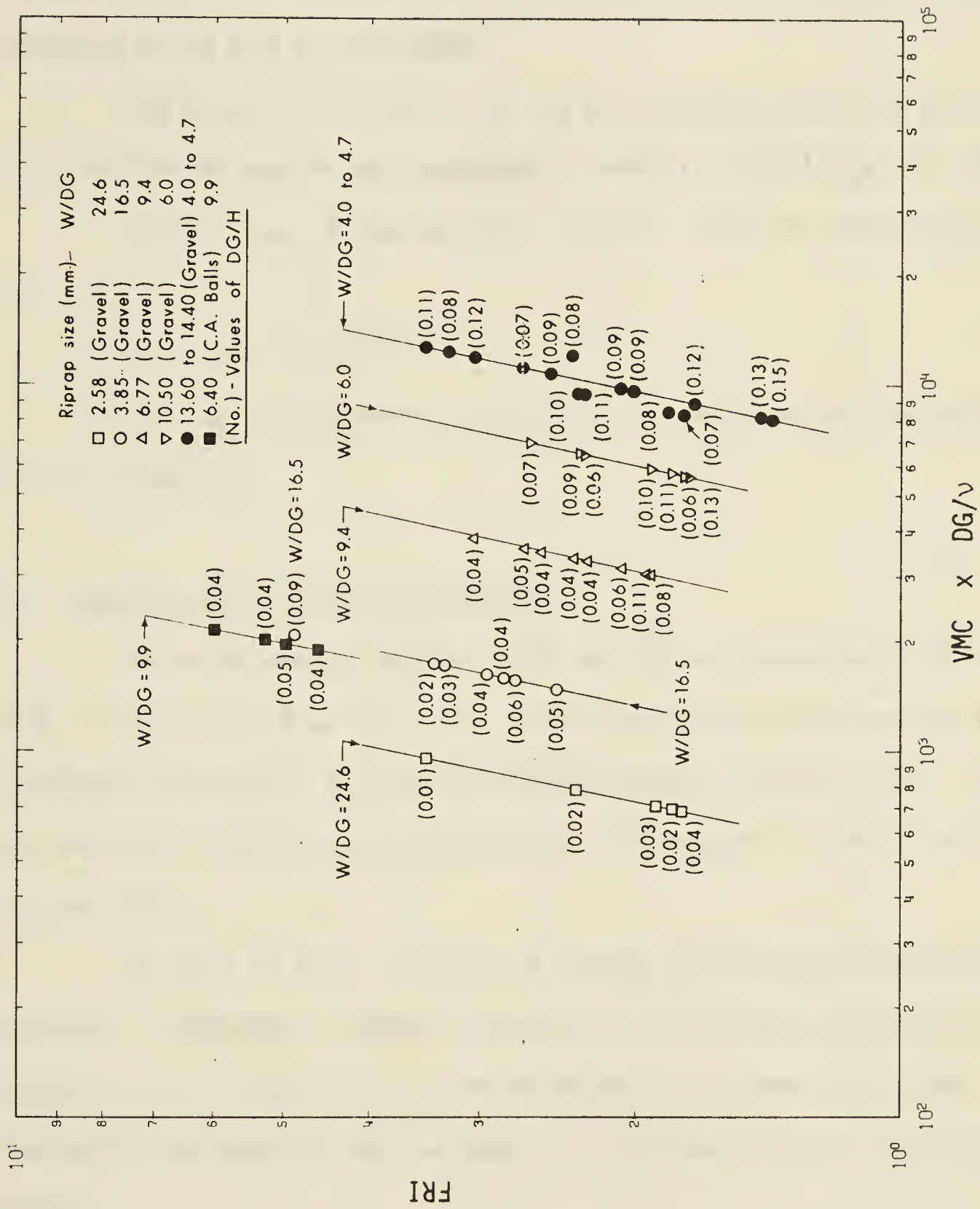


FIGURE 5-1 VARIATION OF VMC x DG/v WITH FRI FOR PIER RIPRAP DATA

fitting line is significant on Fig. 5-2. The general trend on Fig. 5-2 is the decrease in FR1 with the increase in DG/H. Lines representing constant values of $\gamma'_s DG^3/\rho_f v^2$ and W/DG on Fig. 5-2 do not show any systematic trend. Considering the gravels only the slopes of these lines (a to f) are not constant.

From these observations it can be concluded that the scatter on Fig. 5-2 is not due to any systematic trend in $\gamma'_s DG^3/\rho_f v^2$ or W/DG .

The equation of the best fit line for DG/H vs FR1 relationship is:

$$FR1 = 1.14 (DG/H)^{-0.20} \quad (5-5)$$

Table A-2 in Appendix A lists all the statistical parameters for this regression.

5-7 Relationship Between H/DG and FR'

The relationship between H/DG and FR' as suggested by Equation 5-2 is plotted on Fig. 5-3. The data used is from test series A and B. Different points are plotted for each material size and type. Values of the third and the fourth parameter in Equation 5-2 are also given on the plot.

The plot on Fig. 5-3 shows a strong correlation between H/DG and FR'. Regression analysis revealed a correlation coefficient of 0.98, for this correlation. The equation of the best fit line and all other statistical parameters are given in Table A-2 in Appendix A.

Fr' vs H/DG relationship shows that for higher values of H/DG (small materials as riprap) correspondingly higher values of FR' are required to start movement around the pier. This is explained by the

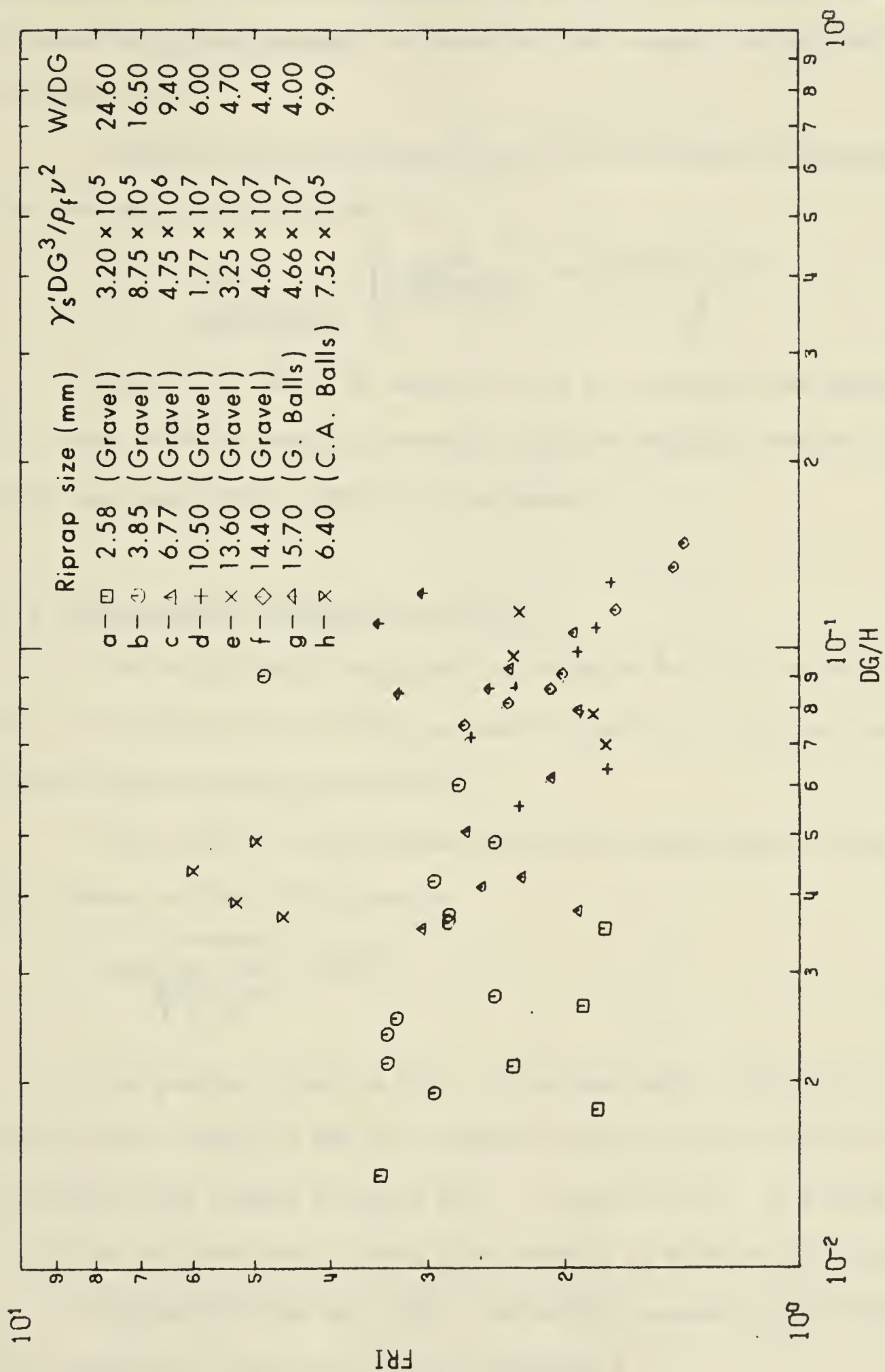


FIGURE 5-2 VARIATION OF DG/H WITH FRI FOR PIER RIPRAP DATA

presence of DG^3 in the denominator of FR' . With relatively small difference in q from material to material, FR' values are primarily effected by DG .

Strong correlation shown by Fig. 5-3 can partly be explained by the presence of H/DG in FR' , as:

$$FR' = \frac{q}{\sqrt{g(SG-1)DG^3}} = \left(\frac{VMC^2}{g(SG-1)DG} \times (H/DG)^2 \right)^{1/2}$$

$\gamma'_s DG^3 / \rho_f v^2$ and W/DG values follow a trend with the material size i.e. with H/DG increasing (material size decreasing compared to depth) W/DG increases while $\gamma'_s DG^3 / \rho_f v^2$ decreases.

5-8 Relationship Between H/DG and FRD

The relationship suggested by Equation 5-3 is plotted in Fig. 5-4. The values of the third parameter $\gamma'_s DG^3 / \rho_f v^2$ and the fourth parameter W/DG are given on the plot.

The scatter is considerably more than that shown in Fig. 5-3. One reason is that FRD is simply

$$FRD = \sqrt{\frac{\rho_f VMC^2}{\gamma'_s DG}} = \sqrt{FR1}$$

The general trend on Fig. 5-4 is that with comparable values of H/DG higher values of FRD are required for initiating motion of bigger size materials around the same pier. Scatter on Fig. 5-4 does not seem to follow any systematic trend with respect to W/DG or $\gamma'_s DG^3 / \rho_f v^2$.

Regression line and other statistical parameters for this analysis are given in Table A-2 of the Appendix A.

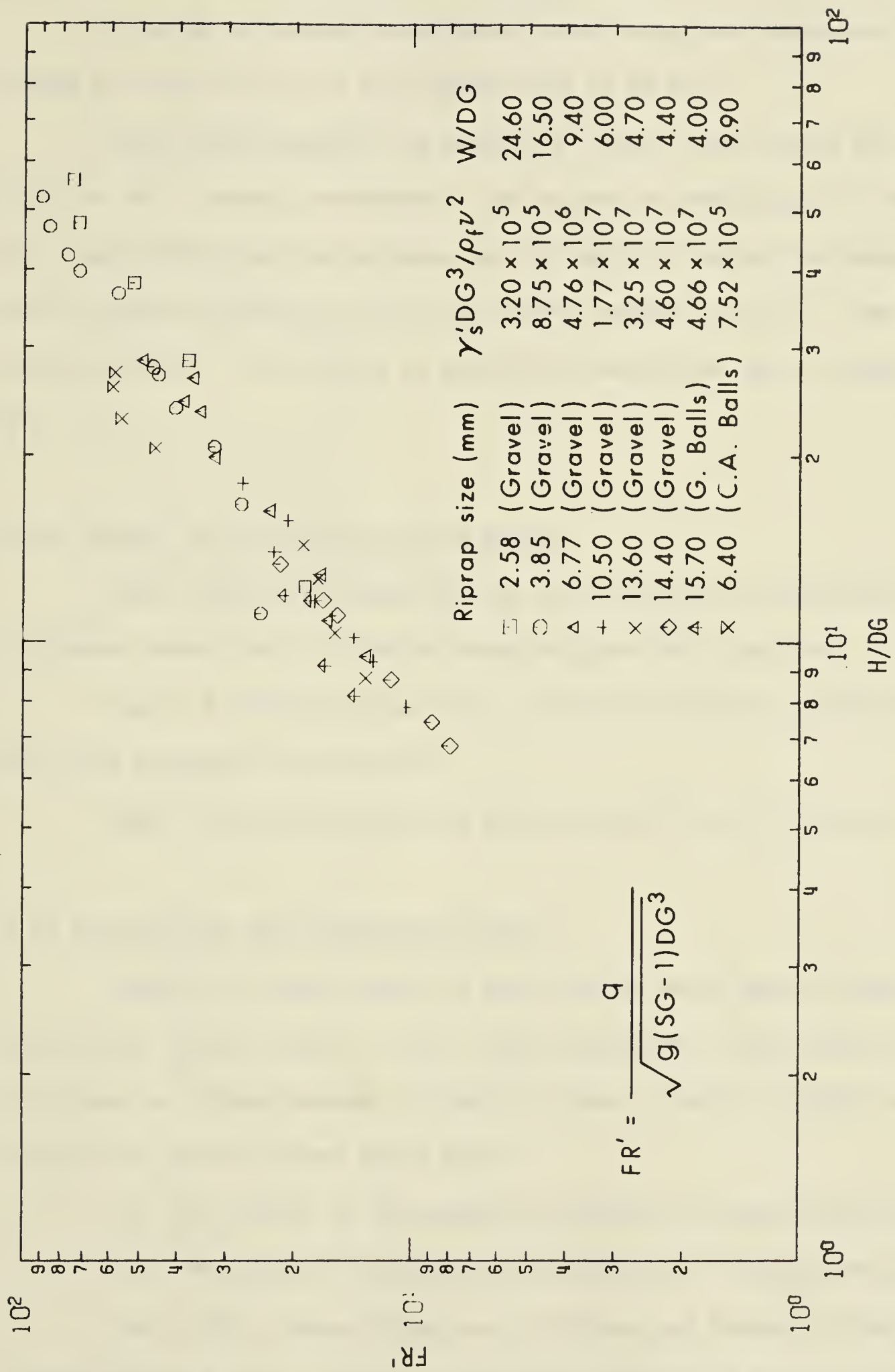


FIGURE 5-3 VARIATION OF H/DG WITH FR' FOR PIER RIPRAP DATA

5-9 Effect of Angle of Attack on Pier Riprap

In order to assess this effect some tests were conducted with a fixed pier size (2.5 in) and riprap size (3.85 mm).

Figure 5-5 presents the results of these tests using FR' and H/DG as the plotting parameters. It is obvious from Fig. 5-5 that for the same H/DG values an increase in the angle of attack decreases the mean velocity required to fail the riprap around the pier. Due to the limited number of the tests no definite trend lines can be drawn on Fig. 5-5.

5-10 Effect of Pier Size on Pier Riprap

Some tests were conducted with 3.85 mm riprap around piers of different width, but of similar shape aligned with the flow.

Fig. 5-6 indicates that the critical velocity does decrease with the increase in pier size.

Table 4-4 contains all the data for the Fig. 5-5 and 5-6.

5-11 Correlation with Comparative Data

There is no known data for pier riprap tests under similar conditions as in this study. Neill (1968) conducted a few tests with coal fractions as riprap around cylindrical piers. Neill's (1968) observations from these limited tests were:

- (a) The effect of flow depth is slight on riprap stability.
- (b) The approach roughness does effect the riprap behaviour.

Shen (1971) quotes flume tests by Maza and Sanchez (1964) on riprap around a pier, but their original publication was not available.

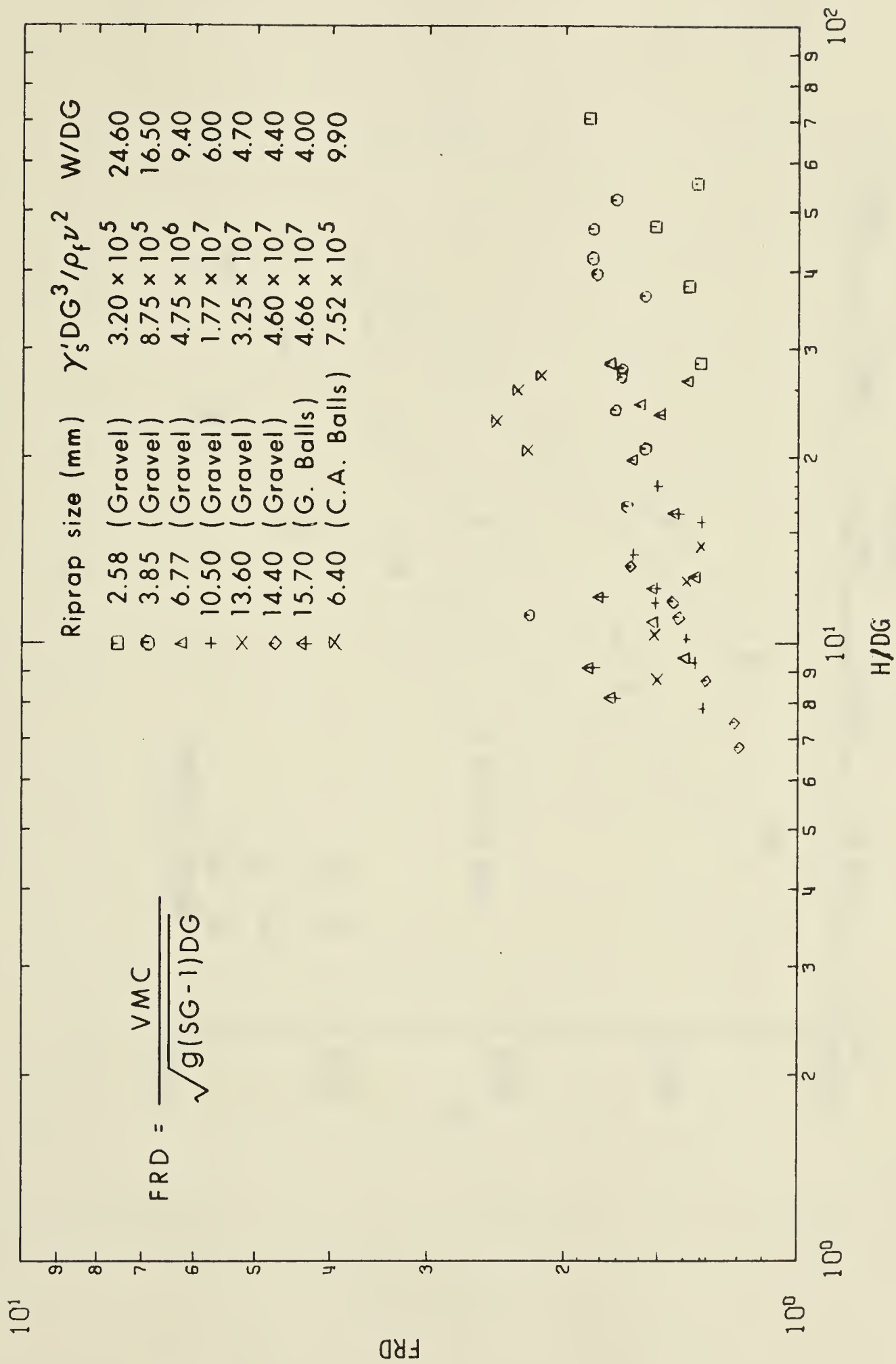


FIGURE 5-4 VARIATION OF H/DG WITH FRD FOR PIER RIPRAP DATA

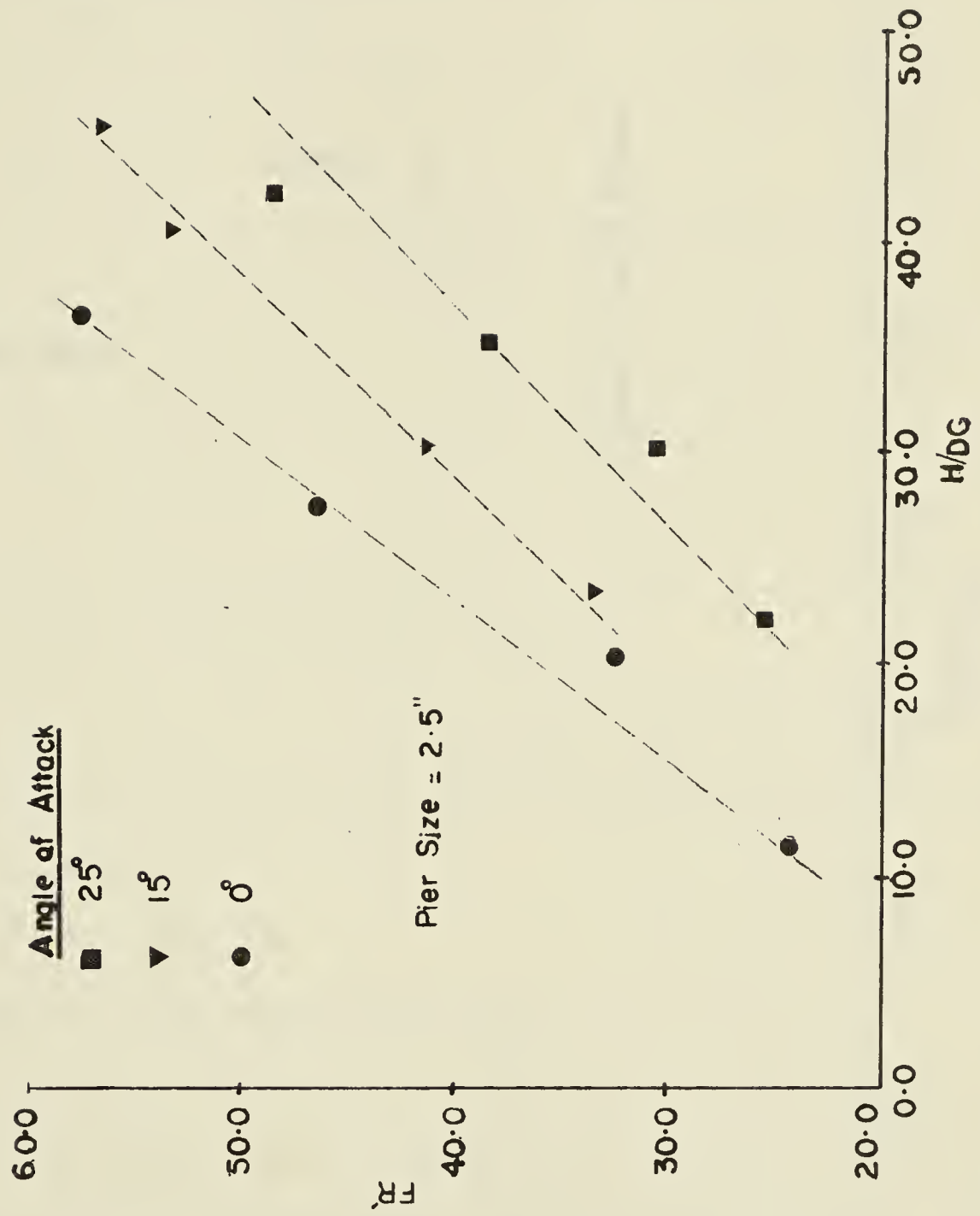


FIGURE 5-5 VARIATION OF H/DG WITH FR' FOR VARIOUS ANGLES OF ATTACK

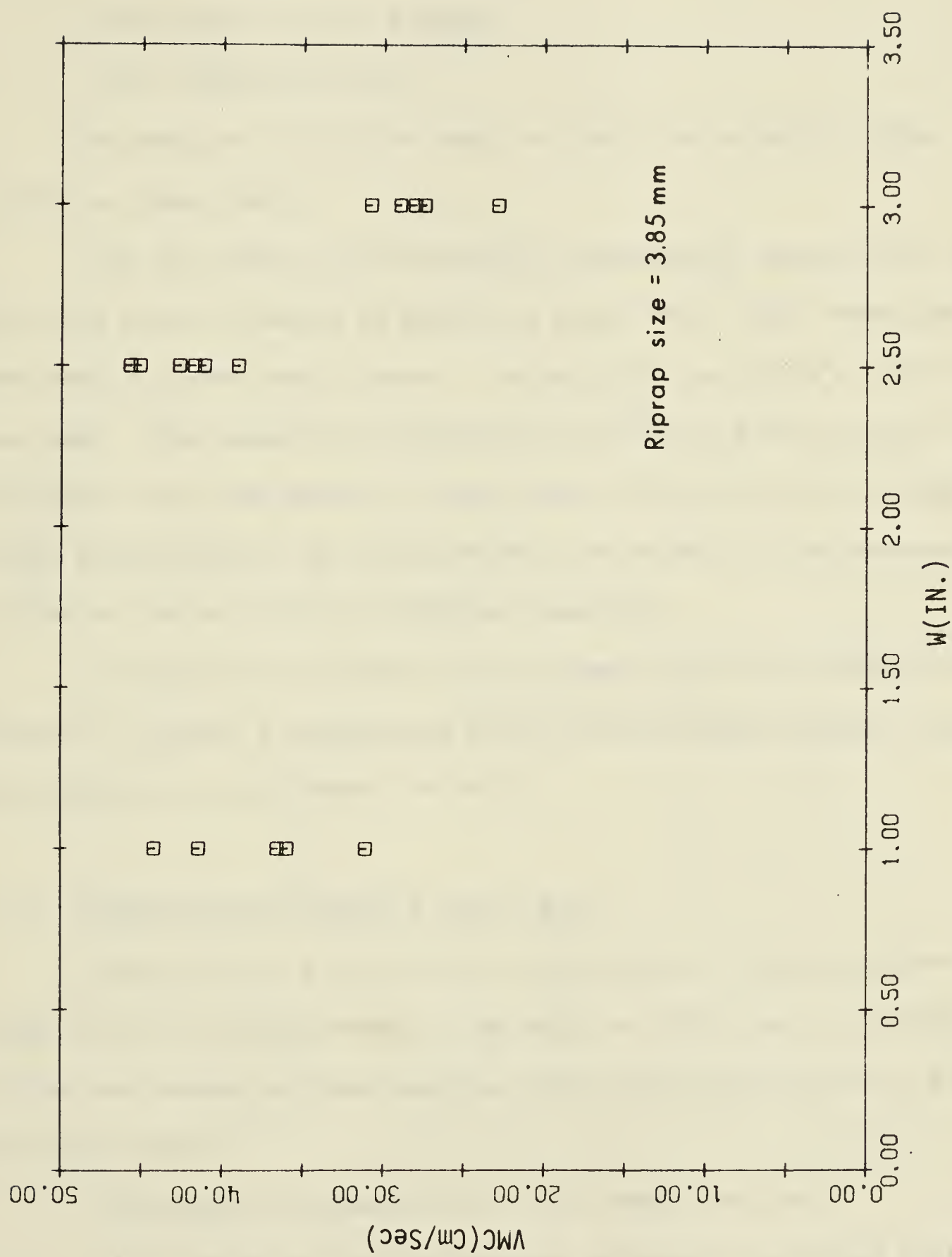


FIGURE 5-6 VARIATION OF VMC WITH PIER WIDTH FOR PIER RIPRAP

The equation from the analysis of data quoted by Shen (1971) is:

$$VMC = 9.55 (DG)^{0.63}$$

where:

VMC = Mean velocity (ft/sec)

DG = Riprap size (ft)

No mention of the flow depth or pier size is made by Shen (1971) in these tests.

The only other course open for a comparative analysis was with the data from initiation of motion on plane beds. This comparison was done with two sets of data. For the first set Neill's (1967) data was used. The reason for using Neill's (1967) data was that his test materials were the same as in this study. This way it was possible to study quantitatively and qualitatively the effects of the presence of a pier on the stability of different materials.

The other set of data used for comparison is by Peterson (1972) which is, in fact a compilation of all the published sediment transport data with no or low charge ($0 < c < 0.2$).

5-12 Comparison With Neill's (1967) Data

Figure 5-7 is a plot of the mean critical velocity against the grain size for the experimental and Neill's (1967) data. Different points are marked for each material type and the depth flow is indicated for each point.

The general conclusions from this comparison are:

- (i) The velocities for movement of gravels and glass balls are much lower in the case of a pier riprap than for the same materials on a plane bed with the same depth of flow.

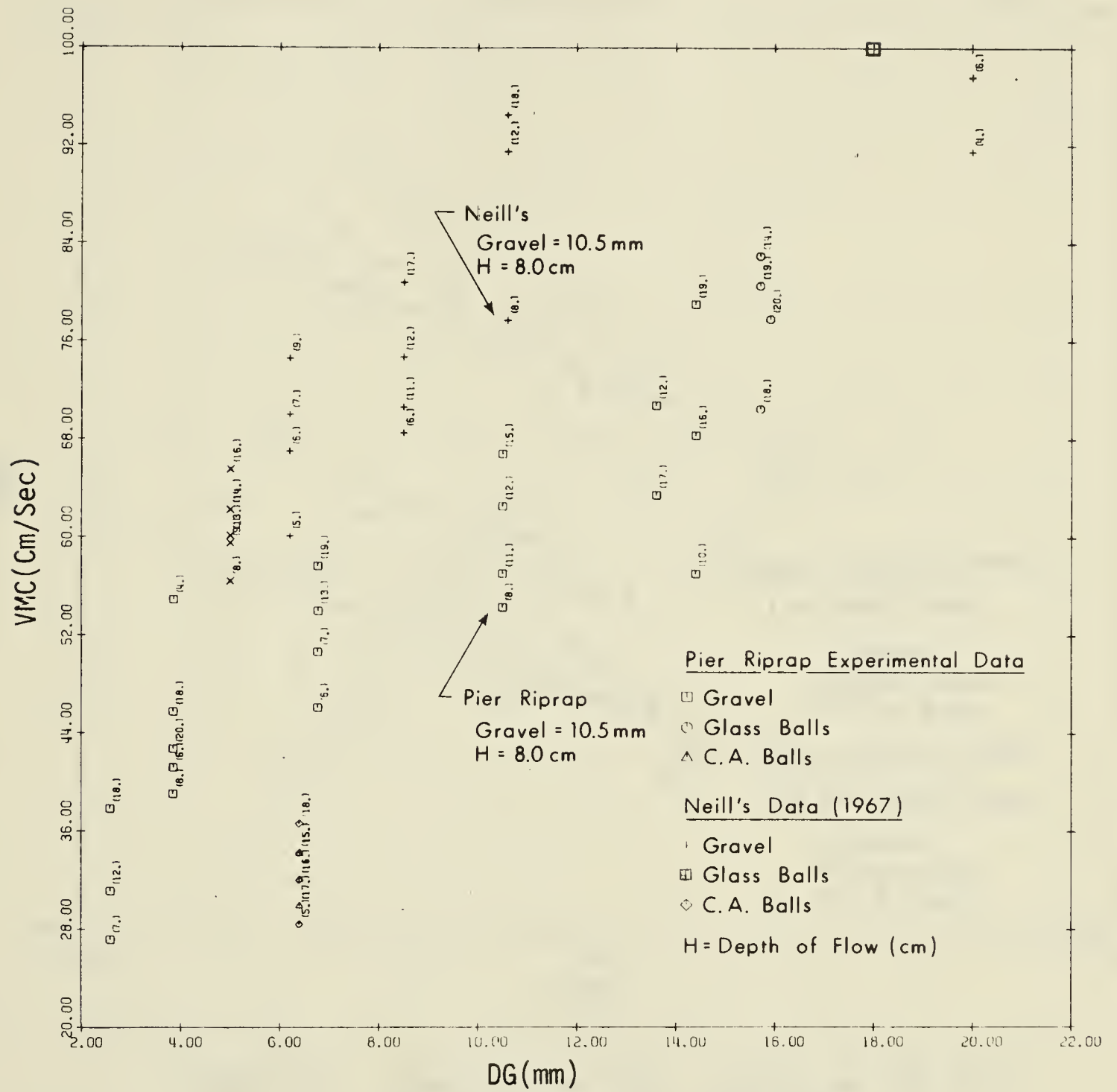


FIGURE 5-7 COMPARISON OF DG VERSUS VMC FOR PIER RIPRAP DATA WITH NEILL'S (1967) DATA

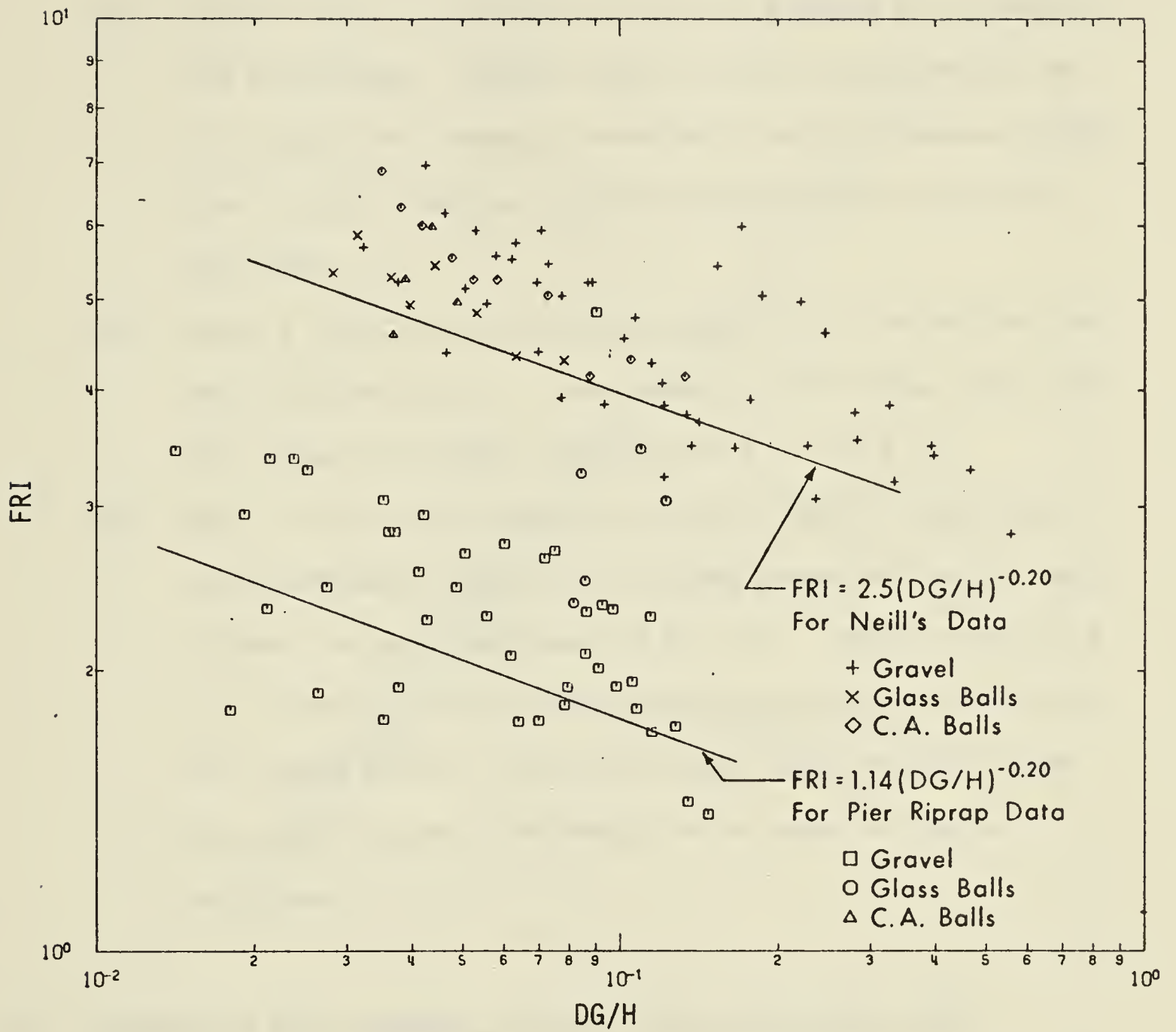


FIGURE 5.8 COMPARISON OF DG/H VERSUS FRI FOR PIER RIPRAP AND NEILL'S (1967) DATA

For example for gravels of 10.5 mm for depths of 8 cm and 10 cm the actual velocity for movement of a pier riprap is about 30% lower than of a plane bed.

- (ii) The data for C. A. balls in both the studies lies almost in the same range. Probably this is due to large depths and low velocities, reducing the effects of the presence of the pier to such a degree that the material fails as if on a plane bed.
- (iii) Neill's (1967) data for 16 mm glass balls lies outside the VMC scale on Fig. 5-7. His values for VMC range from 105-116 cm/sec for depths varying from 7.0 - 15.9 cm.
- (iv) Fig. 5-8 is a non-dimensional plot of Neill's (1967) and the experimental data. The fitting lines for the two sets of data are also indicated on Fig. 5-8. Neill's line is a fit to the lower end of his test band, while the line for the riprap data is a best fit line. Both the lines have the same slope only differing in the value of the coefficient.

5-13 Comparison With Sediment Transport Data (Peterson, 1972)

For evaluating the overall effects of bridge pier presence on the stability of riprap material around it, a comparative analysis was done with sediment transport data (Peterson, 1972). This data is compiled from various sources comprising of some 366 points with charge varying from 0 to 0.20.

Fig. 5-9 is a plot of pier riprap data and the sediment transport data. On the plot are also indicated the values of W/DG ,

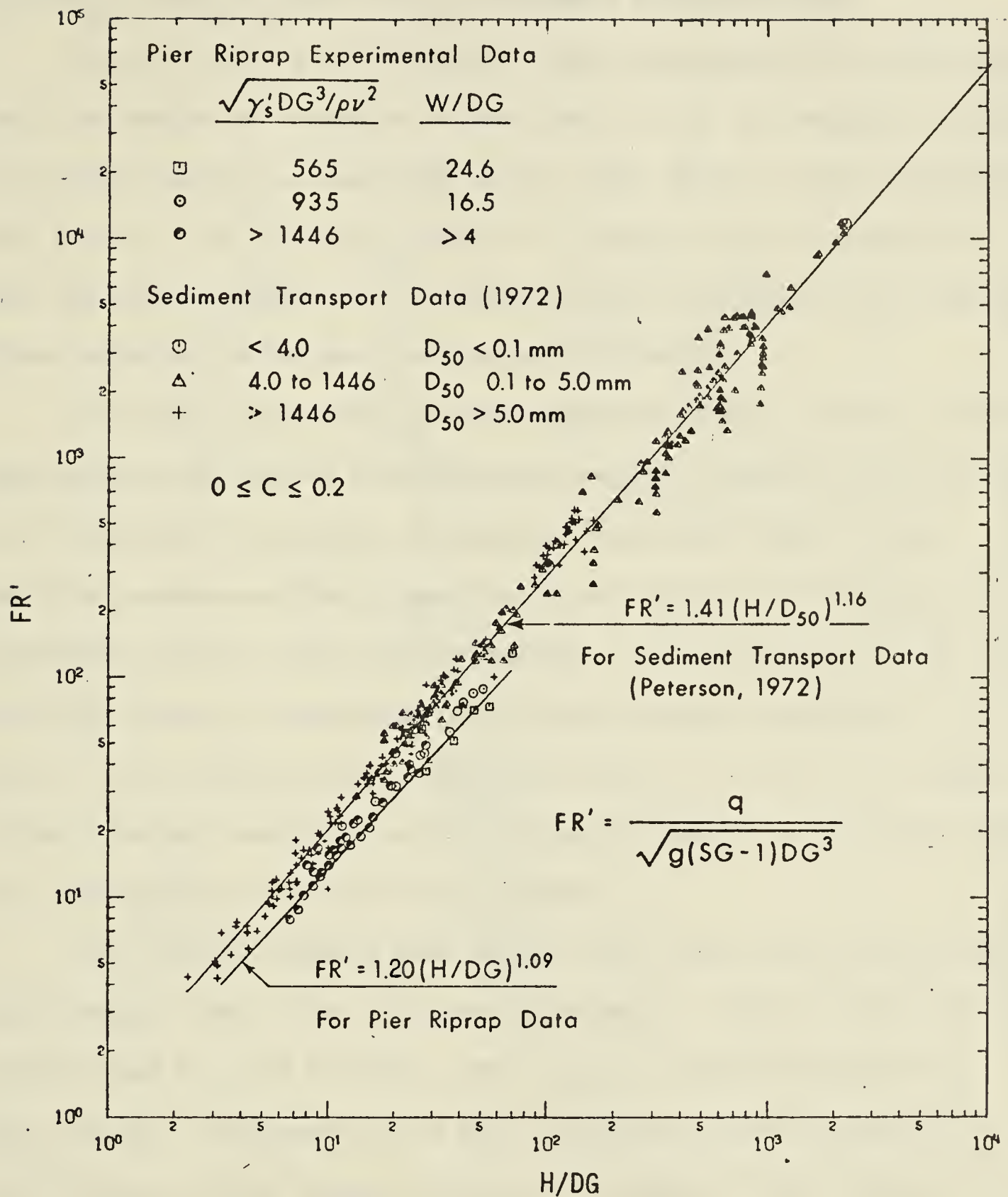


FIGURE 5-9 COMPARATIVE PLOT OF H/DG VERSUS FR' FOR PIER RIPRAP AND SEDIMENT TRANSPORT DATA (PETERSON, 1972)

$\gamma_s DG^3 / \rho_f v^2$ and $\gamma_s D_{50}^3 / \rho_f v^2$ for the sediment transport data.

The plot of Fig. 5-9 indicates that the presence of a pier initiates the motion of riprap at a lower value of FR' as compared to motion on a plane bed for the same H/DG ratio. This shift is quite consistent from size to size (varying $\gamma_s DG^3 / \rho_f v^2$). Although a lot of scatter in both the data is masked by the parameter FR' , the slope of the best fit lines indicated on the plot are not much different.

The above comparison and the comparison of Fig. 5-8 both indicate that despite the complex flow structure around a pier the riprap stability is basically a function of increased velocities near the pier. As the flow accelerates from stagnation around the pier, velocities start increasing and when this velocity reaches a critical value for the riprap their motion is initiated. This picture bears resemblance to the case of a mild constriction. The gross effect of a pier is to impose slightly higher velocities on the riprap as compared to the same riprap on a plane bed for the same depth of flow.

Fig. 5-10 is another plot of the pier riprap data and the sediment transport data using different parameters. This plot shows more scatter than Fig. 5-9 for both sets of data. The observation from Fig. 5-9 that the presence of a pier initiates motion of particle at lower values of mean channel velocity as compared to some material on a plane bed, is further confirmed.

5-14 Proposed Design Curve for Pier Riprap

Comparative analysis of the previous sections indicates that the effect of the presence of a pier on the riprap materials around it is to initiate their motion at lower values of mean velocity than on a

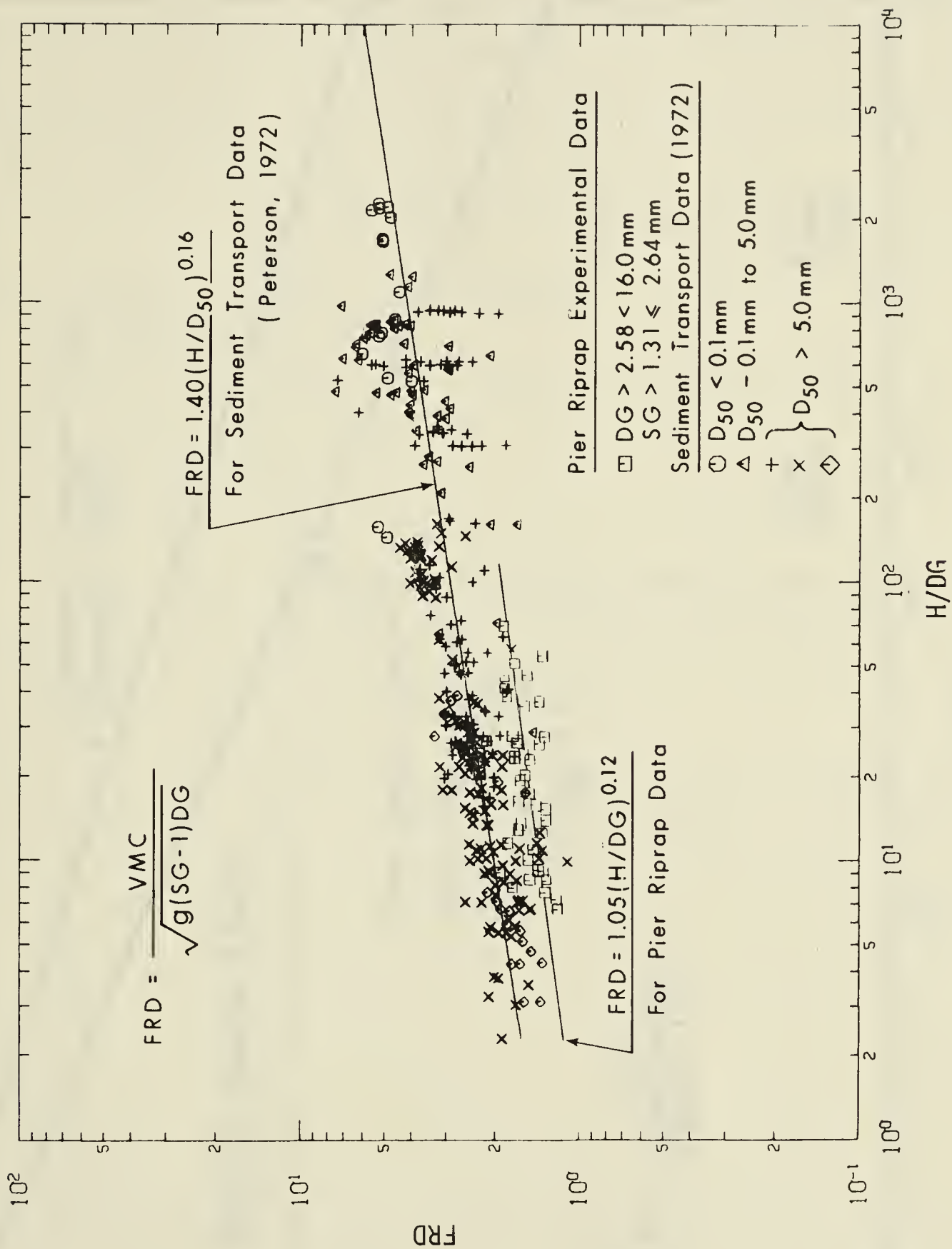


FIGURE 5-10 COMPARATIVE PLOT OF H/DG VERSUS FRD FOR PIER RIPRAP AND SEDIMENT TRANSPORT DATA (PETERSON, 1972)

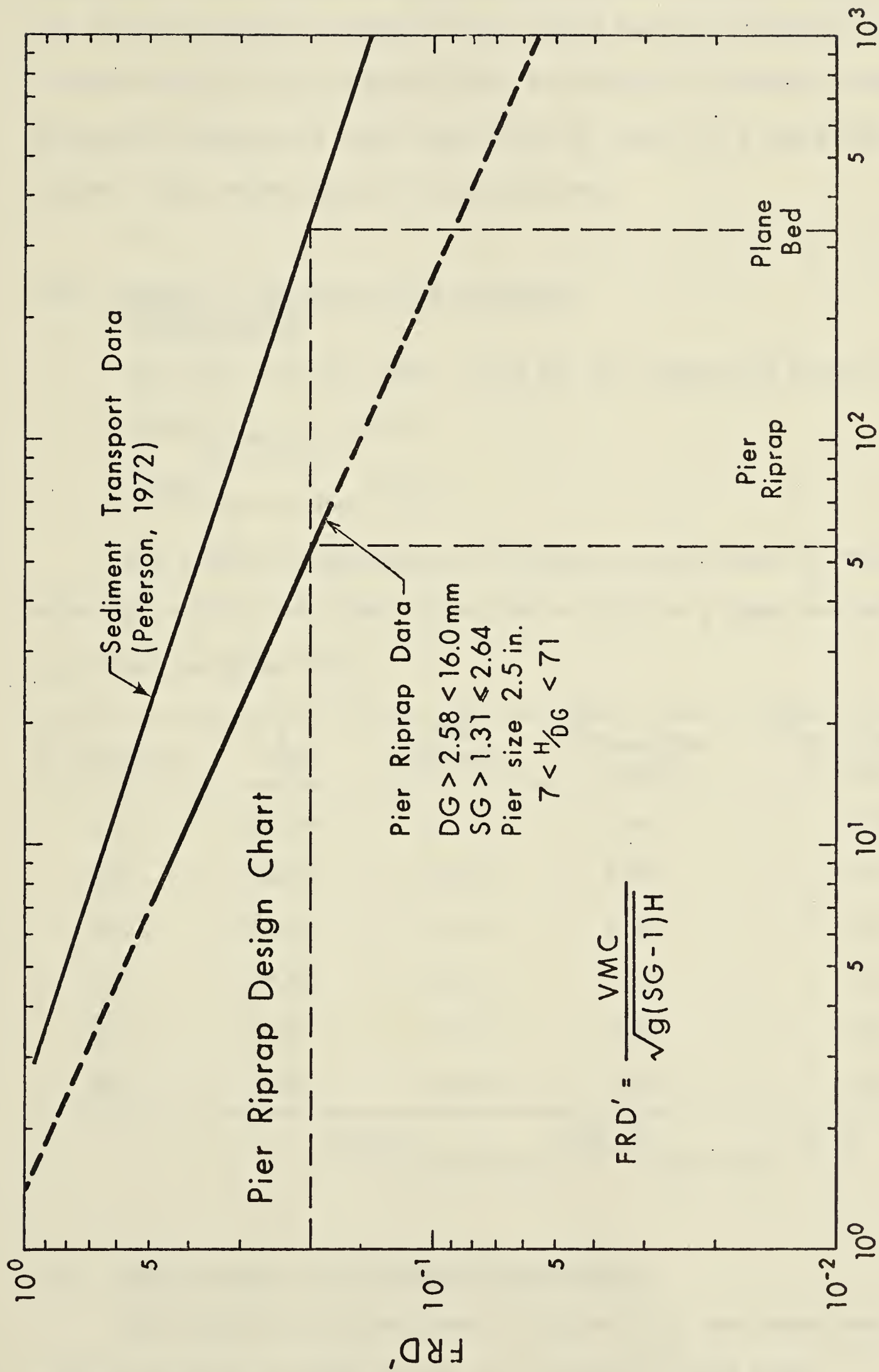


FIGURE 5-11 RECOMMENDED DESIGN CURVE FOR PIER RIPRAP

plane bed. Based on these observations a design curve for pier riprap was derived as shown in Figure 5-11. (Using the data of Figure 5-10). For each value of the parameter FRD' two values of H/DG are obtained giving the comparative sizes that would be stable on a plane bed and around a pier under similar flow conditions.

5-15 Example on the Use of the Proposed Design Curve

From Fig. 5-11 for $FRD' = 0.20$ the two values for H/DG are:

$$(H/DG)_{\text{Plane Bed}} = 330.0$$

$$(H/DG)_{\text{Pier Riprap}} = 54.0$$

For various combinations of the mean velocity and the flow depth with $FRD' = 0.20$, the stable sizes ($SG = 2.65$) on a plane bed and around a pier are as given below.

q (ft ² /sec)	H (ft)	VMC (ft/sec)	$DG_{\text{Plane Bed}}$ (in.)	$DG_{\text{Pier Riprap}}$ (in.)
50	10.59	4.74	0.40	2.36
100	16.80	5.97	0.61	3.74
200	26.6	7.51	0.97	5.91
250	30.9	8.10	1.12	6.88
300	35.0	8.62	1.27	7.80
350	38.6	9.05	1.40	8.60

$$(H/DG)_{\text{Pier Riprap}} / (H/DG)_{\text{Plane Bed}} = 6.10$$

5-16 Limitations of the Proposed Design Curve

For using the design curve of Figure 5-11, the conditions of the geometrical similarity of the prototype to the model

tested must be satisfied. These conditions are:

- (a) The prototype pier should have a round nose similar to the model pier (Figure 1-1).
- (b) $L_p/W \geq 2.5$
- (c) $4 < W/DG < 24.6$

The limitations on flow regime for using the proposed design curve are:

- (d) $FR < 0.6$

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6-1 Summary and Conclusions

The object of the study reported was to develop some criteria for the design of a pier riprap. Tests were conducted on materials of different sizes and specific gravity around a pier of fixed size. Dimensional analysis was used to develop a mathematical model to evaluate these test results.

Plotting and analysis of the data reported indicates that:

- (i) Effect of the presence of a pier, on the stability of riprap around it, is to initiate their motion at lower values of mean velocity than for the same material on a plane bed. As indicated by Fig. 5-9 mean channel velocity can still be used for designing the pier ripraps, but its' value has to be reduced from that on a plane bed as shown in Fig. 5-7.
- (ii) Design of a pier riprap can be based on the design chart derived in Figure 5-11. The limitations on such a design are indicated in 5-16.
- (iii) Tests on relative width of pier effects on the riprap stability indicate that the mean velocity for failure of the riprap decrease with the increase in the relative width of the pier.
- (iv) Tests on angle of attack indicate that the mean velocity required to initiate motion of pier riprap materials decreases with the increase in angle of attack.

6-2 Recommendations

For developing the criteria for pier riprap design further and for clarifying some aspects left out in this study, the following is suggested:

- (i) A series of tests to determine the effects of the relative pier width and the nose shape of the pier on the riprap stability.
- (ii) A series of tests to determine the optimum proportioning parameters (a' , p , t) for a pier riprap.
- (iii) A series of tests to determine the effect of the bed material in the approach channel on the pier riprap stability.
- (iv) A series of tests to determine the proportioning of an apron of equal strength around a pier. An apron of equal strength is one whose material is just stable for the design flow conditions over its entire surface.
- (v) Tests to determine the limiting conditions after which the riprap failure criteria of this study does not apply.

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APPENDIX A

A-1 Introduction

The observed data for pier riprap is analyzed using different correlations from sediment transport, initiation of motion and bridge pier scour. The results of these analyses and their plots are presented in this appendix.

A-2 Mean Velocity and Grain Size Relationship

The mean velocity of flow is plotted against the grain size as shown in Fig. A-1 for the riprap test data of series A and B. Effects of flow depth and material type are differentiated.

The general trend in Fig. A-1 is that the critical mean velocity for gravels increases with the increase in flow depth. C. A. balls require much lower velocities to move them as compared to the nearest gravels for the same flow depth. This probably is due to their different density and the fluctuating pressures around the pier.

Equation of the best fit line for this correlation is:

$$VMC = 20.6 (DG)^{0.43} \quad (A-1)$$

Neill (1967) got a similar equation from his initiation of motion data as:

$$VMC \propto (DG)^{0.47} \quad (A-2)$$

A-2 Relationship Between TU and DG

The force exerted by the flow on the bed particles can usually be correlated to the particle size for incipient condition (ASCE 1966). Such an attempt was made for the experimental data of series A and B. Fig. A-2 is a plot of the critical tractive force ($TU = \gamma HS$) and the nominal diameter of the grains (DG).

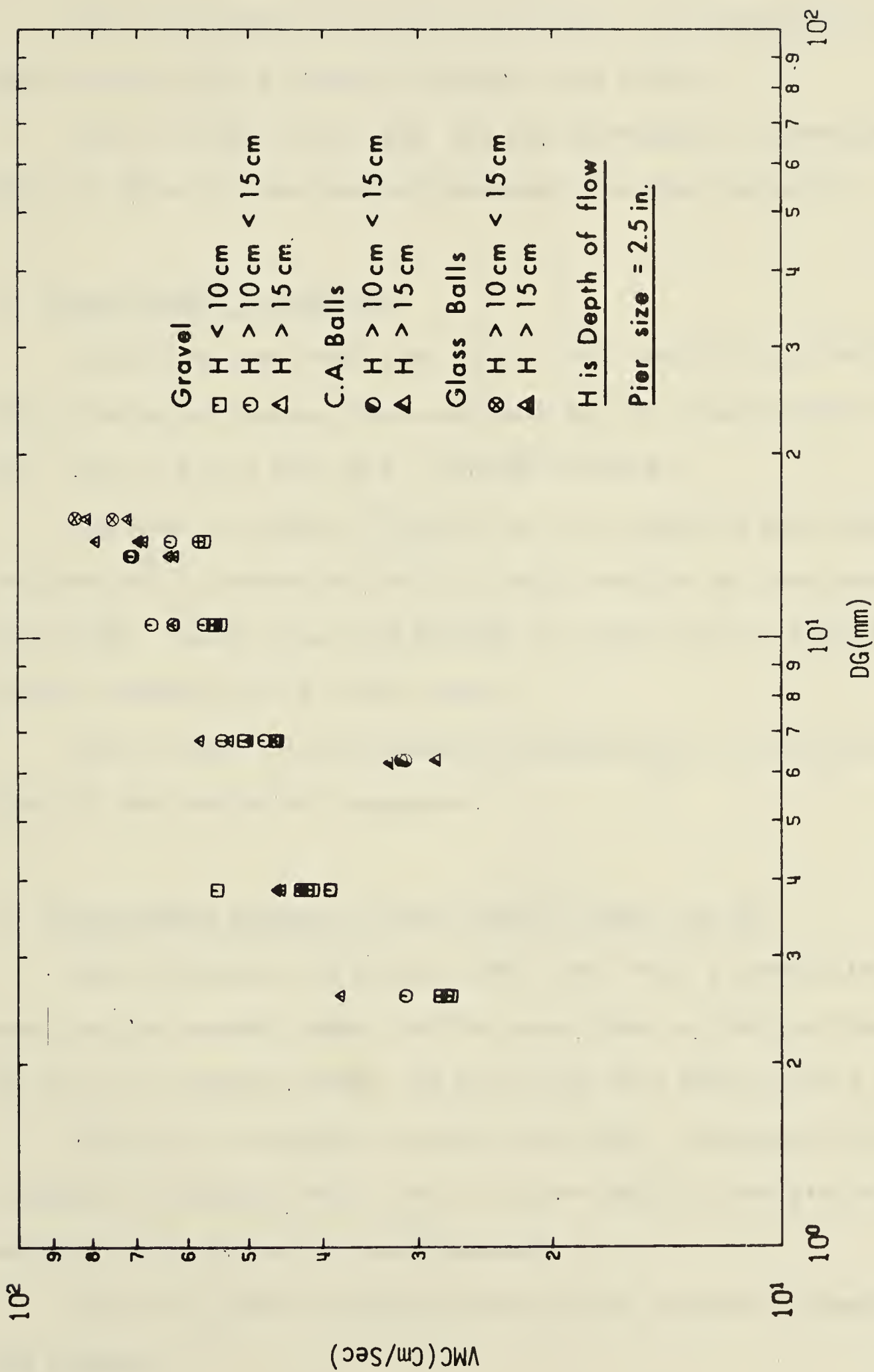


FIGURE A-1 DG VERSUS VMC FOR PIER RIPRAP DATA

The correlation on Fig. A-2 is not good. Apparently the bed shear stress is not a relevant parameter near a pier.

Line of best fit for Fig. A-2 was determined by regression. Table A-2 lists all the computed parameters for this regression.

A-3 Froude Number Correlation

Studies on pier scour (Rao et al., 1971 and Chitale, 1960) have found correlation between the scour depth and the Froude number of the flow. Fig. A-3 is a plot of $F = VMC/\sqrt{gH}$ and DG/H .

The scatter on Fig. A-3 is bad at its lower and upper ends. The lower end is covered by the C. A. balls and the smallest gravel size (2.58). On the upper end are the two points for the glass balls and some random points for the gravels.

Line of best fit was determined by regression and the Table A-2 gives all the statistical parameters.

A-3 Relationship Between the Pier Reynold Number and DG

Shen, Schneider and Karaki (1969) have found a correlation between the pier Reynold number and the scour depth at the pier nose. Fig. A-4 is a similar attempt for the riprap data from series A and B.

There is no consistent trend on this plot. Apparently this plot is similar to plotting VMC vs DG, as in $RN = VMC W/\nu$, the pier size W and the kinematic viscosity are constants.

Table A-2 gives the line of best and the computed parameters for this analysis.

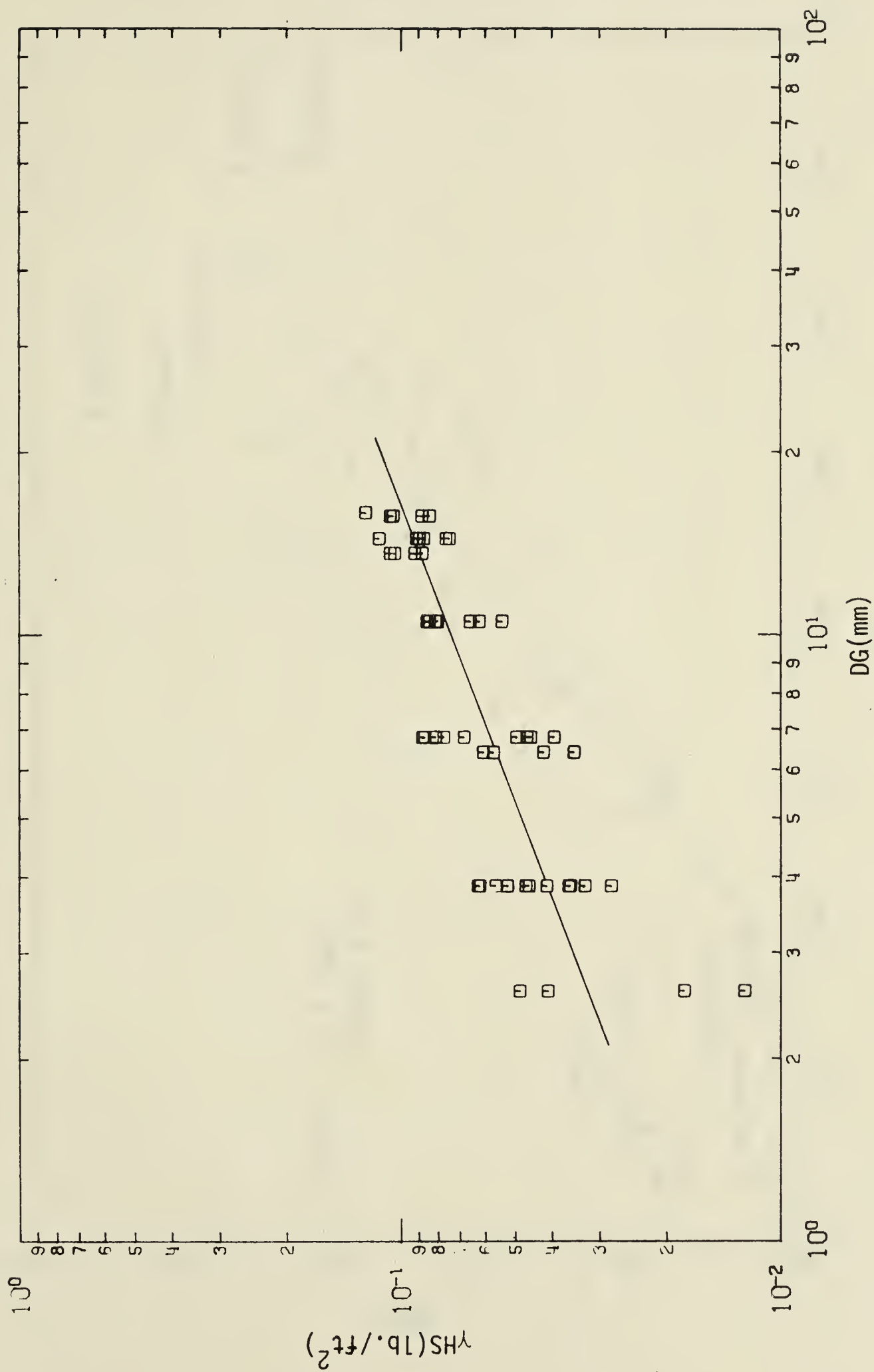


FIGURE A-2 DG VERSUS MEAN TRACTIVE FORCE FOR PIER RIPRAP DATA

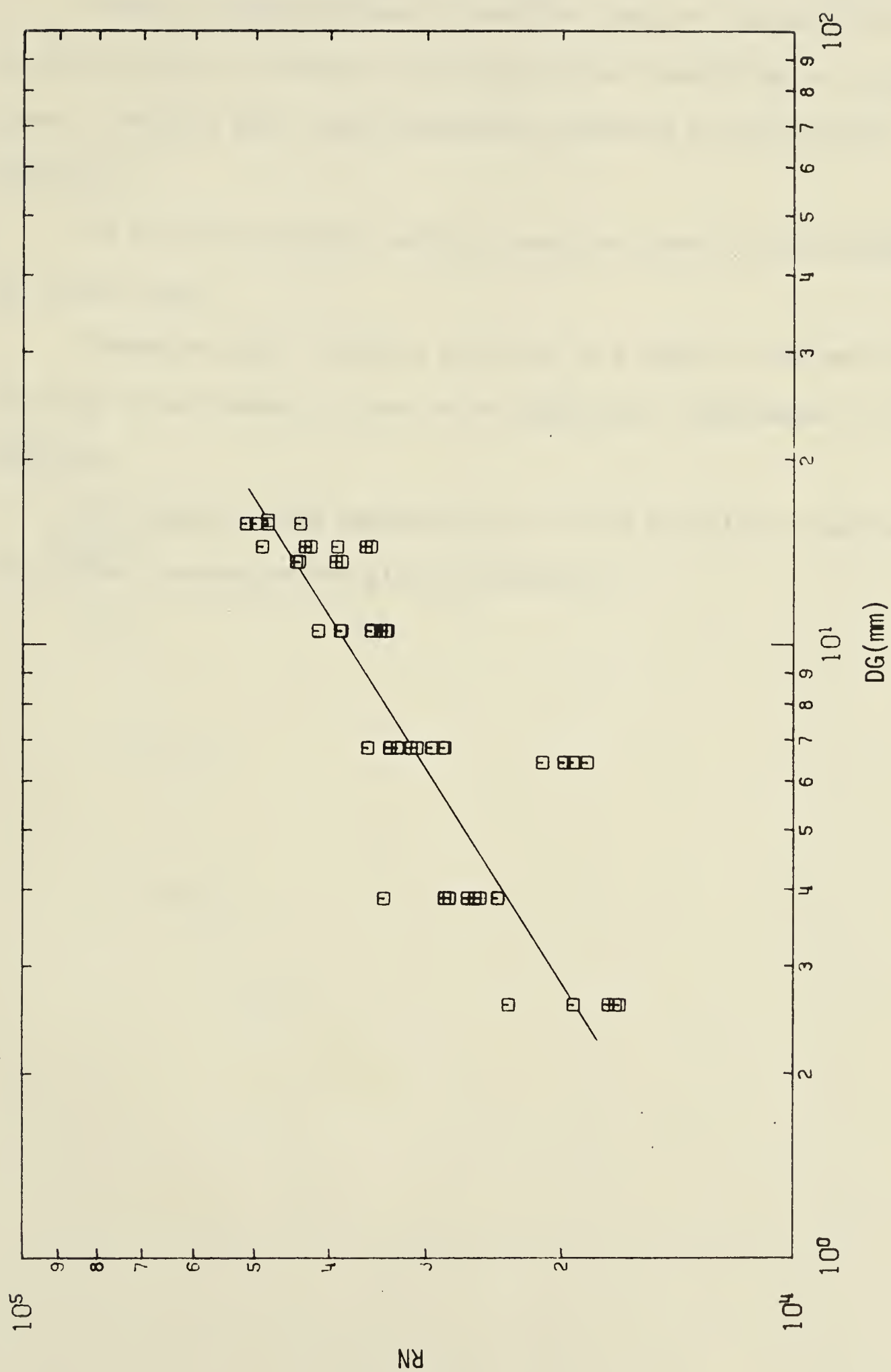


FIGURE A-4 VARIATION OF DG WITH RN(PIER REYNOLD'S NUMBER) FOR PIER RIPRAP DATA

A-4 Critical Velocity Near the Pier and DG Correlation

From test observations, it was felt that the riprap failure around the pier is initiated by the increased velocities at the pier sides. For each test this velocity was measured as explained in Chapter 3.

In Fig. A-5 critical velocity near the pier is plotted against the riprap size.

Comparing Fig. A-5 plots with Fig. A-1 based on the mean critical velocity in the channel, there is no significant improvement in the correlation.

The equation from regression for VPC vs DG relation and the statistical parameters are given in Table A-2.

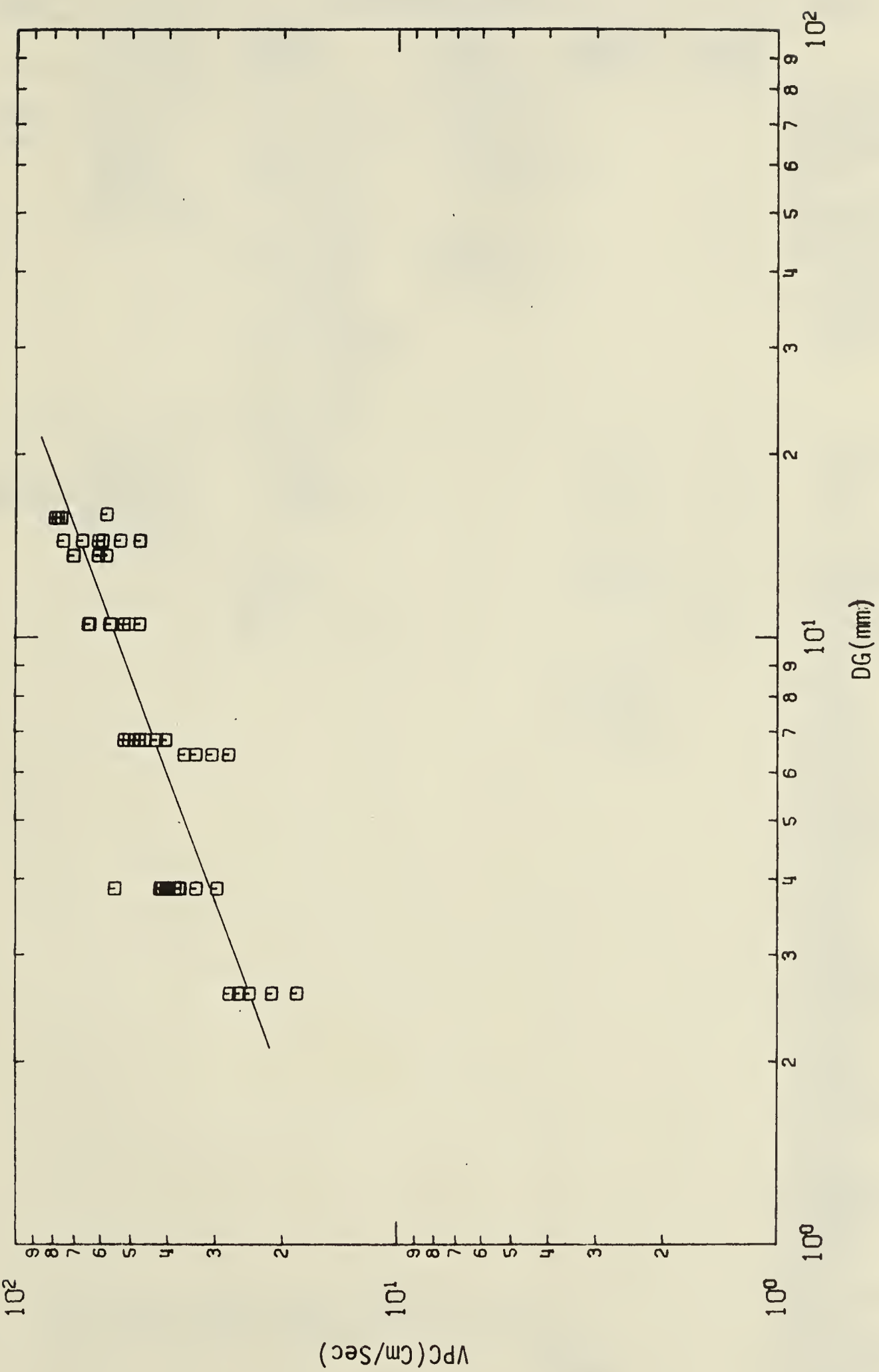
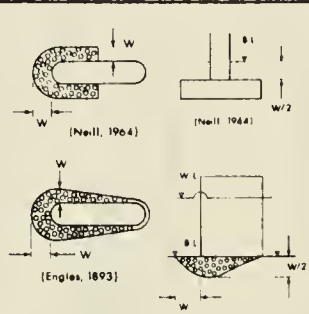


TABLE A-1 SUMMARY OF RIPRAP DESIGN CRITERIA BY DIFFERENT AUTHORS

TYPE OF PROTECTION	RECOMMENDED CRITERIA FOR					REMARKS
	CHARACTERISTIC STONE SIZE	THICKNESS OF RIPRAP LAYER	GRADING OF RIPRAP	EXTENT AND SHAPE OF RIPRAP	LOCATION OF THE RIPRAP	
ON PLANE BED	$D_{50} = 100 \text{ lb. stone}$ or 3 x the size that moves at high flood [Blench, 1961]	$t = 1.5 \times D_m$	$\frac{D_{15} \text{ Filter}}{D_{85} \text{ Base}} < 5$	$4 < \frac{D_{15} \text{ Filter}}{D_{85} \text{ Base}} < 20$		$D_{15} = 15\%$ of the material is finer than this size by weight. Base refers to the material to be protected $D_m = \text{Average rock size}$ $Q = \text{discharge (CFS)}$
		or $t = \text{Max. Rock Size}$ [ASCE, 1948] $t < 2.75 \times D_{50}$ $> 1.50 \times D_{50}$ [Burgess et al 1966] $t = 2 \text{ largest stone size}$ [California Highways, 1960] $t = 0.06 Q^{1/3}$ [Inglis, 1949]	$\frac{D_{50} \text{ Filter}}{D_{50} \text{ Base}} < 25$ [Posey, 1955]			
FOR PIER PROTECTION	$D_{50} = 5 \times \text{the size that moves on bed without the pier}$ [Varzeliots, 1960]	$t = 6.5 D_m$ [Sousa-Pinto, 1959] $t = \text{width of the pier}$ or $t = 3 \times D_R$ whichever is larger [Maza et al, 1969]	$\frac{a'}{d_p} = k \frac{(d_s - p)}{d_p}$ [Sousa-Pinto, 1959]		$t = \text{thickness of the riprap layer}$ $D_m = \text{characteristic size of the riprap layer such as } D_{50} \text{ or } D_{85}$ $D_R = \text{Diameter of the uniform riprap}$ $W = \text{width of protection around the pier}$ $t = \text{thickness of the pier}$ $BL = \text{streambed level}$ $WL = \text{water level in stream}$ $a' = \text{lateral extent of riprap layer}$ $d_p = \text{diameter of the pier}$ $d_s = \text{depth of scour without protection}$ $p = \text{depth at which riprap layer is located below the streambed}$ $K = \text{Experimental constant} = 1.8 \text{ for circular piers}$ $W^* = \text{wt. of the stone (in lbs)}$ $b_i = \text{width of the pier (ft.)}$ $D_p = \text{thickness of the protective layer (ft.)}$	

$$\frac{Q}{b_i} = 2.3g^{1/2} \times \left(\frac{W^*}{\gamma_s}\right)^{1/6} \times \left[\left(\frac{\gamma_s}{\gamma}\right) - 1\right]^{1/2} \times b_i^{1/2} \times d^{2/3} \times D_p^{1/4}$$

[Inglis, 1949]

TABLE A-2
REGRESSION ANALYSIS SUMMARY

CORRELATION TYPE	DEPENDENT VARIABLE Y	INDEPENDENT VARIABLE X	CORRELATION X vs Y	b	a	S.E*	EQUATION OF BEST FIT LINE	REMARKS
$Y = ax^b$	VMC	DG	0.84	0.43	20.6	0.07	$VMC = 20.6 DG^{0.43}$	VMC = Critical Mean Velocity (CM/SEC) DG = Nominal Dia. of Grains (MM)
$Y = ax^b$	FR1	$\frac{DG}{H}$	-0.37	-0.199	1.14	0.13	$FR1 = 1.14 \left(\frac{DG}{H}\right)^{-0.199}$	$FR1 = \rho_f \frac{x VMC^2}{\gamma_s x DG}$ $\frac{DG}{H}$ = Relative Depth
$Y = ax^b$	TU	DG	0.78	0.70	0.15	0.15	$TU = 0.15 (DG)^{0.70}$	H = Depth of Flow (CM) TU = Critical Shear Stress = γHS S = Water Surface Slope
$Y = a+bx$	$\frac{DG}{H}$	FR	0.82	0.205	-0.029	0.02	$\frac{DG}{H} = 0.029 + 0.0205FR$	FR = $\frac{VMC}{\sqrt{gH}}$
$Y = ax^b$	DG	RN	0.85	1.67	0.0000002	0.14	$DG = 0.0000002(RN)^{1.67}$	RN = $VMC \times \frac{W}{\nu}$ W = Pier width ν = Kinematic viscosity
$Y = ax^b$	VPC	DG	0.89	0.53	0.52	0.07	$VPC = 0.52(DG)^{0.53}$	VPC = Bottom velocity near the pier at failure
$Y = ax^b$	$\frac{VMC}{\sqrt{g(SG-1)DG}}$	H/DG	0.20	0.05	1.05	0.071	$\frac{VMC}{\sqrt{g(SG-1)DG}} = 1.05 (H/DG)^{0.120}$	
$Y = ax^b$	FR'	H/DG	0.98	1.05	1.10	0.04	$FR' = 1.10 (H/DG)^{1.07}$	$FR' = \frac{q}{\sqrt{(SG-1)g DG^3}}$ q = Q/B
$Y = ax^b$	$\frac{VPC}{\sqrt{gDG}}$	H/DG	-0.09	0.02	1.86	0.07	$\frac{VPC}{\sqrt{gDG}} = 1.86 (H/DG)^{-0.02}$	

* S-E - Standard Error of the Estimate

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